VOLUME 80 SEPARATE No. 384

PROCEEDINGS

AMERICAN SOCIETY OF CIVIL ENGINEERS

JANUARY, 1954

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

DISCUSSION OF PROCEEDINGS - SEPARATES

153, 169, 175

HYDRAULICS DIVISION

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

Current discussion of papers sponsored by the Hydraulics Division is presented as follows:

This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N.Y.

DISCUSSION OF
CHARACTERISTICS OF FIXED-DISPERSION CONE VALUES
PROCEEDINGS-SEPARATE NO. 153

EDWIN W. MURPHY[3].—Two 48-in. valves of the type discussed in this paper were installed by Messrs. Howell and Bunger, at El Vado Dam in Chama, N. Mex., in 1935. They are now operating under a load of 140 ft. Since that time valves have been developed in sizes ranging from 4 in. to 108 in. in diameter and for heads varying from 55 ft to 700 ft.

Hydraulic engineers who are faced with the problem of controlling discharge of sluice water under head need information from field tests. The data presented in this paper, therefore, are of great value to designers.

Observations from several installations are reported, and the results compared so that the conclusions drawn from these data do not represent an isolated condition meeting only specific requirements. With tests conducted on five valves, whose sizes range from 78 in. to 96 in., the findings in this paper present a general picture of performance.

The designers and manufacturers of any new product must depend first on their calculations, and next on results of model tests to evaluate the finished article and to predict its performance. Reports of tests from the field under actual working conditions are needed to compare with the original assumptions so that proper correction factors may be determined. A concise method of calibrating the actual discharge of the valve under varying head and gate openings is included in the paper. Eq. 7 gives the discharge coefficient for any condition of head. Fig. 7 illustrates how the discharge coefficient varies from the closed position to the maximum open position of the valve. The efficiency of the valves is shown to increase as the size of the valves increases although all designs are homologous.

The point of maximum discharge for the valve (Fig. 7) is not at its full open position but at a point slightly ahead of it. When the valve was first being developed, the stroke was made slightly longer than was believed necessary. This was done because many installations are made to control discharge of waters impounded during flood stage and gradually discharged later so as not to cause property damage downstream. Under these circumstances, the valve must be used to draw down the reservoir level from a maximum head to the zero point. Under conditions of extreme low head this additional stroke was to provide more opening and to discharge faster. The Fontana valve, when tested under a gross head of 295.8 ft, reached its point of maximum discharge with the gate opened at 92.5% of its full stroke (Fig. 7, Fontana high head). When tested under a gross head of 32.0 ft, this valve shows its best discharge at 98.5% of its full stroke (Fig. 7, Fontana low head). These results indicate that the valve stroke should be made sufficiently long to take care of all conditions. In the field, the maximum desired open position can be easily determined, and through a simple adjustment of a limit switch the operating mechanism will limit the travel of the gate to that point. A vibration is set up by the discharge when the cylinder gate is opened beyond the

[3] Head Structural Engr., S. Morgan Smith Co., York, Pa.

point where the lip of the sleeve is in contact with the jet (see under the heading, "Discharge Characteristics: Operational Characteristics of Valves.") When the lip is moved beyond this point there is a "make and break" action between it and the jet. This action is believed to cause the vibration. Since opening the valve wider will eliminate this action and the vibration, but will add nothing to its discharge, the logical solution is to limit travel to a point just ahead of the spot where the discharge lip is clear from the discharge jet.

The valve under discussion was designed primarily as an energy dissipater and free-discharge valve. Through its action of throwing its jet out in a hollow, cone-shaped, expanding spray, the valve obtains the maximum resistance from the air and thus dissipates a large part of the energy carried in the water. Consider the figures for the Fontana valve when operating under a head of 295.8 ft and at its point of maximum discharge: Under these conditions it is spilling 4,600 cu ft of water every second. In terms of hydraulic energy—

$$HP = 0.1134\, Q\, H \qquad (8)$$

—it is found that 155,000 horsepower (HP) of energy is to be controlled. This energy, unless dissipated at once, would destroy anything in its path. To visualize this enormous force, consider a 12-in. valve discharging under an 850-ft head. This valve throws out a mountain of spray approximately 40 ft in diameter. The air resistance encountered absorbs nearly all energy and dissipates it. The large spread of the jet eliminates any necessity of digging a huge pothole at the base of the dam for an energy absorber.

Many locations do not have an unlimited space for the valve jet to spread out and be broken up by air resistance as does the one already described. It is necessary then to provide some means of controlling the spread of the jet and restricting the air resistance. Although this action does defeat the energy dissipater function, it is possible to reach a compromise. The first attempt to do this was to place a separate cylinder, or hood, around the valve. This has been done on an installation in South America in which an 8-in. valve is discharging through such a hood and operating under a 700-ft head. This valve shows a marked decrease in the spread of the discharge jet. However, the water has considerable energy remaining in it as it leaves the end of the hood, and could seriously erode the tailrace were the channel not properly protected.

The Fontana installation (Fig. 3) is a further development of this principle, in that it discharges into a tunnel 15 ft in diameter, leading to the river downstream from the dam. Baffles are placed in the tunnel to break up the jet and to destroy part of the energy. It is evident that, when the free expansion of the jet is restricted in this manner, large quantities of air will be drawn into the tunnel along with discharge from the valve. In the early development stages some method was needed to determine how large an air inlet should be to admit sufficient air to supply this demand. With what information was then available, and by studies from a laboratory test model, the manufacturers suggested that the vent area be made at least equal to the square of the diameter of the valve. From the field tests on Fontana project it is illustrated that the wind velocity through the access tunnel at full valve open position is slightly less than 20 miles per hr. The access tunnel provides a

total vent area of 60 sq ft. A vent 3 ft in diameter furnishes an additional area of 7 sq ft. Assuming that the velocity through this vent will be the same as through the tunnel, the valve is drawing air at approximately 2,000 cu ft per sec. The maximum discharge does not require as much air as other gate positions. The addition of baffles in the path of the valve jet also modifies the air requirements.

To engineers who are studying proposed installations, and to the manufacturers and designers of this type of valve, the presentation of these findings provides a means for studying and comparing the results with the assumed design and to base calculations on a somewhat sounder footing. These findings were based on a particular case and, unless all features are duplicated, the same results cannot be expected in other cases. When test data become available from the field operation of a piece of equipment, comparison with original model tests is imperative. Through such a comparison it is possible to obtain new correction factors through which a proper step-up between model and prototype can be made.

Laboratory tests do not always present a true picture of how the prototype will perform. Many engineers hold differing opinions as to proper coefficients to use in rating a model test against expected field performance. This difference in opinion applies in many fields other than the study of hydraulics. It appears that much research work remains to be done along these lines.

The valve under discussion is of a very simple construction, with only one moving part contacting the water. Its construction eliminates, almost entirely, the effects of water load or hydraulic unbalance in opening or closing the cylinder gates. Almost all the operating force required is needed to overcome the mechanical friction of the stuffing box and necessary gearing. On one installation, where the valve discharges downward at a 30° angle and with cylinder sleeve operated by an oil pressure servomotor, the operator found that slightly less pressure, as calculated, was required to close against full head than to open. This observation indicates that a fraction of weight of the moving parts is greater than hydraulic unbalance.

With the elimination of moving parts, sources of vibration are not present, and the entire valve operates smoothly without any of the destructive vibrations set up in most free-discharge controlling devices—a spouting jet of water under high head can be throttled without disturbing a coin balanced edgewise on the body of the valve.

Rodolfo E. Ballester[4].—In Argentina there have been installed six fixed-dispersion cone valves of the same type as the valve described by the authors. However, the valves in Argentina differ from Howell-Bunger valves in that they are operated by levers instead of screws.

Table 2 lists the installations. There are two valves at each dam. All the valves discharge into the open air, making unnecessary any air-demand provisions in the surrounding structures, such as those described by the authors under the heading, "Characteristics of Associated Structures."

---

[4] Prof. of Applied Hydraulics, Univ. of Buenos Aires, Buenos Aires, Argentina.

TABLE 2.—Fixed-Dispersion Cone Valves Installed in Argentina

Dams	Valve Diameter		Minimum Head		Maximum Head	
	Meters	Inches	Meters	Feet	Meters	Feet
San Roque........	1.524	60.0	3.0°	10	28.0	92
La Viña..........	1.100	43.3	3.0°	10	56.0	184
Cruz del Eje......	1.100	43.3	3.0°	10	22.5	74

In order to control the spreading and direction of the jets, deviations from the standard valve design have been used successfully at two dams. The first of these adaptations is the reduction of the central angle of the cone valve; the other is the introduction of an angle between the axis of the valves and the axis of the conduits. The authors may be able to add information or opinions concerning similar unusual installations.

The results of model tests using a scale of 1:10 have been described by the writer,[5] and some comparisons might be made with the results given by the authors. From the results of these tests, a table and a graph were prepared for the operation of the prototype valves. This procedure is in contrast to the method of field observation and prototype study used by the authors.

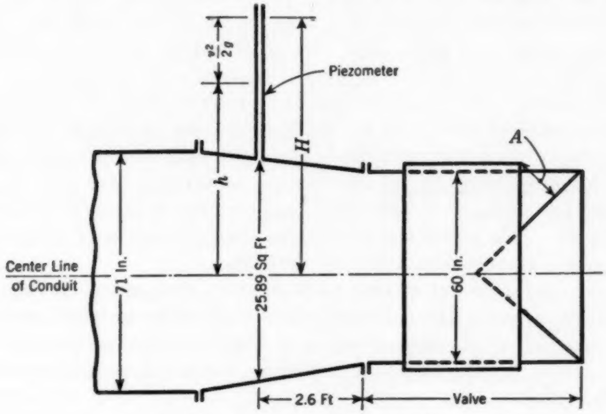

Fig. 14.—Fixed-Dispersion Cone Valve

The discharge into the conduit at a short distance upstream of the valve is given by Eq. 7. During the model tests, the discharge Q and the static pressure h were measured for different positions of the sleeve. The cross section of flow A seems to have been taken by the authors as the cylindrical opening left by the sleeve. However, in the tests described by the writer, a section was selected normal to the surface of the cone and bounded by the edge of the sleeve, and A was defined as this area. This section shows contraction along

[5] "Válvulas para la regulación de la descarga de los embalses de San Roque, La Viña y Cruz del Eje," by R. E. Ballester, *La Ingeniería*, Buenos Aires, Argentina, *No. 856*, February, 1946, pp. 91–101.

the sleeve and no contraction along the cone. Fig. 14 illustrates the meanings of symbols used in the computations.

If the area of the cross section at the piezometer is w, and v is the velocity, $\frac{v^2}{2g} = \frac{Q^2}{2g\,w^2}$ and the final equation is

$$Q = \frac{1}{\sqrt{1 - \frac{C^2}{w^2} A^2}} \; C A \sqrt{2\,g\,h} \ldots\ldots\ldots\ldots (9)$$

The coefficient of discharge for openings of from 22% to 79% of the sleeve course was nearly constant and equal to 0.79. For an opening of 88% of the course, C increases to 0.81. For small openings of 12% of the sleeve course, the coefficient increases to 0.85. This peculiar result may have been the result of an observational error, because an error of 0.2 millimeter (0.008 in.) in the measure of the depth of the normal opening in the model causes an error of 3.9% in the computation of the area of flow.

The valves have been in operation since 1943, and no troubles have been recorded. Unfortunately, no special measurements have been made in the prototypes to check the model experiments, and the graph that was prepared has been used for regulating the discharges. Procedures such as the authors describe would be of value in checking this graph.

The data in the authors' paper are not sufficient to permit the computation of the coefficient of discharge using the area of flow normal to the cone. If the authors would make this interesting computation, the results should yield a coefficient of discharge having less variation than that shown in Fig. 7.

T. T. Siao.[6]—The method of rating valves which the authors have presented is both interesting and informative. Eqs. 1 and 3 are fundamental to a study of valves. The equations indicate that Q is proportional to $\sqrt{\Delta P}$. The combined form of Eq. 6 indicates that a few measurements of Q will suffice, through the rating of $\sqrt{\Delta P / H_G}$ against the sleeve travel s, so as to establish the relationship between these quantities. The various rating curves in Figs. 6 and 8 show that a very systematic set of results is obtained. Only in the curves for the Watauga Dam (Figs. 8(*c*) and 8(*d*)) do the discharge measurements show significant departures from the main trend, perhaps because the condition of the constancy of the K-value for the two valves at the Watauga Dam has not been fulfiled. It cannot be overemphasized that, in order to yield a constant K-value, the two piezometer taps must be located far enough from the valve opening so that the flow pattern between and around the taps will undergo practically no change due to alteration of valve opening. The two taps can actually be located anywhere, in so far as the K-value obtained is constant and the value of ΔP is sufficiently large.

The coefficient of discharge used by the authors varies considerably with the valve opening, as seen in Fig. 7, because the area A in Eq. 7 is not the effective area of efflux. A discharge coefficient based on the area of the opening would be more significant. It is possible to derive, by analytical means, a coefficient of discharge of the latter type for comparison with the observed results.

[6] Research Associate, Iowa Inst. of Hydraulic Research, State Univ. of Iowa, Iowa City, Iowa.

The hydraulician has found that a special type of flow embodied in the formation of a jet from a container of simple geometric form can be analyzed by the method of conformal mapping. Although this method has been developed for two-dimensional flow only, the work of Hunter Rouse, M. ASCE, and A. Abul-Fetouh[7] and J. S. McNown, M. ASCE, and E. Y. Hsu,[8] A. M. ASCE, among others, shows that the results obtained for two-dimensional flows can often be applied with good accuracy to the corresponding three-dimensional flows—at least for a bulk characteristic such as the discharge coefficient. According to Mr. McNown, the discharge coefficient of a cone valve is subject to this type of analysis, and a comparison between the results obtained with those observed by the authors in Fig. 7 is relevant to the authors' purpose in presenting the study.

Fig. 15 is a definition sketch of the comparable two-dimensional flow. Revolution of the boundaries EAB and DC about AB gives the boundaries of

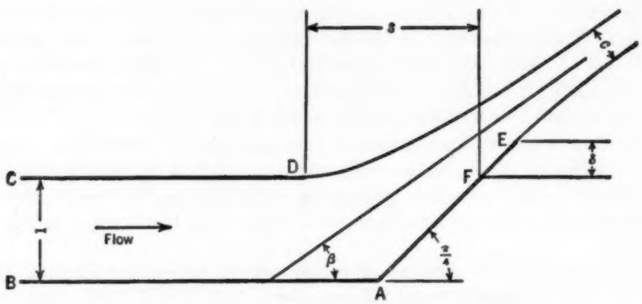

FIG. 15.—SKETCH OF A TWO-DIMENSIONAL JET

a cone valve. It is assumed that there is no loss of energy. By means of a series of mathematical transformations, the flow is changed into that resulting from an idealized source and sink. The relationships between the final width c, the ultimate angle of deflection β, the dimension δ, and the sleeve travel s for the plane flow may be expressed in two equations, as follows:

$$
\begin{aligned}
-\frac{\pi\,\delta}{c} &= (\cos\beta - \sin\beta)\log_e\left[\tan\frac{1}{2}\left(\frac{\pi}{4}+\beta\right)\right] \\
&\quad + (\cos\beta + \sin\beta)\log_e\left[\tan\frac{1}{2}\left(\frac{\pi}{4}-\beta\right)\right] + \pi\cos\beta \\
&\quad + \frac{1}{2}\left(c+\frac{1}{c}\right)\log_e\frac{1+\sqrt{2}\,c+c^2}{1-\sqrt{2}\,c+c^2} + \left(\frac{1}{c}-1\right)\arctan\frac{\sqrt{2}\,c}{1-c^2} \qquad (10a)
\end{aligned}
$$

[7] "Characteristics of Irrotational Flow Through Axially Symmetric Orifices," by Hunter Rouse and A. Abul-Fetouh, *Transactions*, ASME, Vol. 17, 1950, p. 421.

[8] "Application of Conformal Mapping to Divided Flow," by J. S. McNown and E. Y. Hsu, *Proceedings*, Midwest Conference on Fluid Dynamics, J. W. Edwards, Ann Arbor, Mich., 1951.

TABLE 3.—CHARACTERISTICS OF THE TWO-DIMENSIONAL

Characteristic	Sleeve Travel s						
	0	0.0944	0.1889	0.3778	0.5680	0.7619	0.9655
Flow width c	0	0.05	0.1	0.2	0.3	0.4	0.5
Ratio $\frac{c}{s}$	0.530	0.530	0.529	0.529	0.528	0.525	0.518
Angle β, in degrees	42.04	42.04	42.04	42.02	41.99	41.92	41.79
Ratio $\frac{s}{D}$	0	0.0472	0.0945	0.1889	0.2840	0.3810	0.4828
Coefficient C_1	0	0.1	0.2	0.4	0.6	0.8	1.0
Coefficient C_2	0.530	0.530	0.529	0.529	0.528	0.525	0.518

and

$$\frac{\pi s}{c} = 2 \cos \beta \log_e \tan \frac{\beta}{2} + 2 \sin \beta \log_e \left[\tan \frac{1}{2}\left(\frac{\pi}{4} - \beta\right)\right]$$

$$+ \pi(\cos \beta + \sin \beta) + \left(\frac{1}{c} + c\right) \log_e \frac{1 + c}{1 - c}$$

$$+ 2 \left(\frac{1}{c} - c\right) \arctan c \qquad (10b)$$

The two arctan values are to be taken between 0 and $\pi/2$. For simplicity of nomenclature the width of the approaching flow has been taken as unity; this simplification is the same as expressing each of the other dimensions in their ratio to this width. For any given values of δ and s, the two equations can be solved for β and c by trial—or, more directly, any given values of β and c can be substituted into Eqs. 10, and δ and s computed. Eqs. 10, for the special case described in the authors' study for which $\delta = 0$, give values of s, c, c/s, and β, as arranged in Table 3. The discharge coefficient is obtained by dividing the discharge Q by the product of the flow area A and $\sqrt{2 g H}$. Evidently, A can be taken either as the cross-sectional area in the valve body, as the authors have done, or as the area uncovered by the sleeve travel; the corresponding coefficients of discharge are quite different, since A is constant in the former case and variable in the latter. For the two-dimensional flow the two coefficients of discharge are simply c and c/s, provided there is no loss of energy.

A logical approach to the adaptation of these results to the determination of the discharge coefficient for the corresponding three-dimensional flow is to assume that corresponding ratios between the initial and the final area of the jet are the same in each case (as was found for the orifice[7]); that is,

$$\frac{c}{s} = \frac{[w(2 \pi r)]_{ult}}{s(2 \pi \overline{BC})} \qquad (11)$$

in which w is the thickness of the three-dimensional jet at a certain point; r is the distance from the point to the axis AB; $\overline{BC}$ is the width of the approaching flow; and $[w(2 \pi r)]_{ult}$ is the ultimate value of $w(2 \pi r)$. The two coeffi-

Sleeve Travel s						Characteristic
1.1880	1.4490	1.7887	2.3290	2.8369	∞	
0.6	0.7	0.8	0.9	0.95	1.0	Flow width c
0.505	0.483	0.447	0.386	0.335	0	Ratio $\frac{c}{s}$
41.62	41.30	40.82	40.11	39.66	39.19	Angle β, in degrees
0.5940	0.7245	0.8944	1.1645	1.4185	∞	Ratio $\frac{s}{D}$
1.2	1.4	1.6	1.8	1.9	2.0	Coefficient C_1
0.505	0.483	0.447	0.386	0.335	0	Coefficient C_2

cients of discharge, from Eq. 11, are

$$C_1 = \frac{[w(2\,\pi\,r)]_{\text{ult}}}{\pi\,\overline{\text{BC}}^2} = 2\,c \qquad (12a)$$

and

$$C_2 = \frac{[w(2\,\pi\,r)]_{\text{ult}}}{s(2\,\pi\,\overline{\text{BC}})} = \frac{c}{s} \qquad (12b)$$

Table 3 also contains, for the cone valve, the values C_1, C_2, β, and the sleeve travel-valve diameter ratio $\frac{s}{D} = \frac{s}{2\,\overline{\text{BC}}} = \frac{s}{2}$. The C_1-values are comparable to those used by the authors in presenting their test results in Fig. 7, since both are defined in the same manner. The values of C_1, C_2, and β have been plotted against s/D in Fig. 16, in which the authors' test results for C_1, from Fig. 7, are included. Since the assumption shown in Eq. 11 is based on the similarity of pattern of the axially symmetrical flow to that of the corresponding plane flow, it is expected that the smaller the ratio s/D the better the assumption will be, and vice versa. For large values of s/D, Eq. 11 becomes absurd; in fact, it cannot be valid at all for $c > 0.5$, since C_1 cannot be greater than unity. For this reason, only values corresponding to $C_1 < 1.0$ are plotted. It can be seen that there is close agreement between the test results and the analysis for s/D up to 0.25. For $(s/D) > 0.25$ the test results start to fall below, as this type of analysis of the axisymmetric flow can no longer be expected to be accurate for large values of s/D. The true C_1-curve must be tangential to the horizontal line $C_1 = 1$ at $(s/D) = \infty$, and also to the curve $C_1 = 2\,c$ at $(s/D) = 0$.

Even for $(s/D) < 0.25$ the test results are systematically slightly smaller than those computed. The explanation could be that the boundary layer which develops along the valve wall causes the total discharge under a certain head to be less than that which would be attained under the same head if the velocity of approach were truly constant. In contrast to C_1, C_2 varies only slightly with s/D. It is of interest to note that, for the valid range $0 < (s/D) < 0.25$, a constant theoretical value of $C_2 = 0.530$ can be adopted. As to the practical values of C_2, converted from the test results for C_1 by dividing the latter by $4\,s/D$, it can safely be regarded as a constant equal to 0.521 for

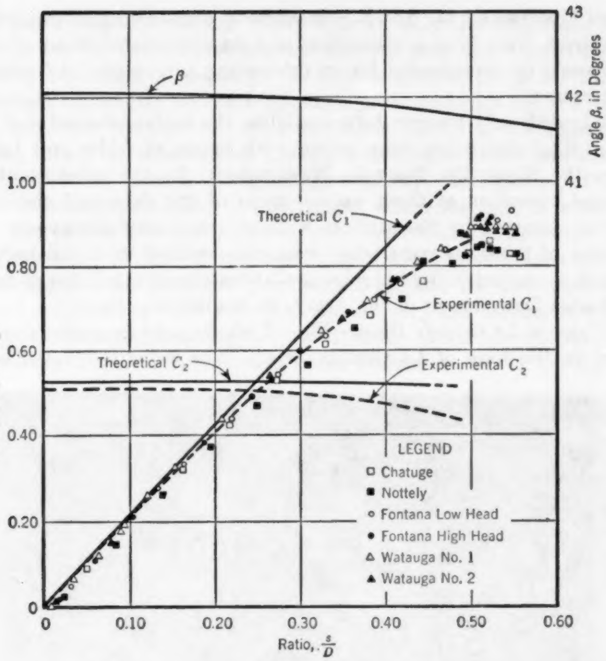

FIG. 16.—CHARACTERISTIC CURVES FOR A CONE VALVE

$(s/D) < 0.25$. Therefore, the discharge coefficient C_2, in comparison to C_1, has certain merits in so far as it not only is conventional, but also remains practically constant for small (s/D)-ratios.

Regarding the deflection angle β, the writer would like to point out that Fig. 13 does not seem to be accurate, since it shows the jet to follow the 45° direction throughout its course. Actually, there will always be some difference between the angle of the cone and that of the jet. For the corresponding two-dimensional jet, the final angle of deflection is between 39.19° and 42.04° for all possible sleeve travels. Even though it is impossible to predict the exact value of β for the cone valve, the range is likely to be approximately the same.

It has been shown that the results of theoretical analysis are directly useful for values of s/D less than approximately 0.25. The theoretical method is also applicable for cone valves of angles other than 45°. Modifications of the design of the angle of the cone valve could therefore be studied analytically. The discharge coefficient and the final angle of deflection β, which give an immediate evaluation of the force exerted on the cone, should be among the primary considerations. Although experimental research would be necessary, a theoretical analysis would undoubtedly provide, at very little cost, a basis for a comparison of the significant characteristics. A judicious combination of both theory and experiment provides the required results.

Verne Gongwer,[9] M. ASCE.—In view of the scarcity of published data on the subject, from both a theoretical and an operational point of view, the authors should be commended for an interesting and substantial contribution to the literature.

In 1941, with only meager data available, the writer adopted and installed two 66-in. fixed-dispersion cone valves with hoods at Alder and La Grande dams, on the Nisqually River in Washington. In the subsequent testing, gaging, and operation of these valves, some of the data and characteristics which were reported by the authors were independently discovered and verfied. Some of these characteristics were also verified in model tests on this type of valve conducted in the hydraulic laboratory of the Corps of Engineers, United States Department of the Army, at Bonneville, Ore.

The Valve at La Grande Dam.—Fig. 17 shows a 66-in. valve (during construction) in the base of La Grande Dam. The valve is located below the

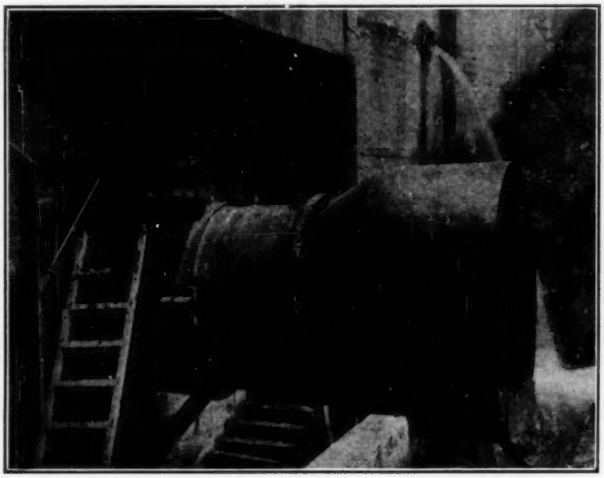

Fig. 17.—La Grande Dam (Washington) Valve During Construction

bucket of the overflow "ski-jump" spillway, and is fitted with a steel hood. Originally, it was intended to surround the valve with a concrete, steel-lined box, or to confine it with concrete "side-blinders." However, the manufacturer suggested a simple conical-cyclindrical hood of unstiffened $\frac{1}{4}$-in. plate, 8 ft in diameter, bolted directly to the valve sleeve. Model tests were performed which gave apparently satisfactory results. This valve was identical with the Alder Dam valve but was to operate under less maximum head (187 ft). There is no air venting of the valve or hood.

The Valve at Alder Dam.—As at La Grande Dam, this valve was intended to supply water intermittently to the next powerhouse downstream in case of outage of the turbines. However, owing to the restricted availability of

[9] Construction Engr., U.S.N., Civ. Engr. Corps, Twenty Nine Palms, Calif; Formerly Chf. Engr., Construction, Dept. of Public Utilities, Tacoma, Wash.

materials and equipment during World War II, only the 40,000-kw unit was released for the La Grande powerhouse and the two Alder Dam units were withheld for more than 18 months. As a consequence, the Alder Dam valve had to operate constantly for more than two years, at heads up to a maximum of 265 ft, whenever the reservoir level was below the lip of the spillway.

Starting with a full reservoir, as the dry season approached and the storage level dropped slowly toward the lip of the spillway, some pitting of the ¼-in. steel hood developed just below the 30° angle, where the conical frustrum was bolted to the end of the valve sleeve, and one small pinhole went entirely through the steel. Failure of the hood would have been very serious since the spreading jet could not have been permitted, and the new 40,000-kw La Grande unit might have been forced to run on very low stream flow, with a 200,000-acre-ft reservoir standing full but useless. After hurried studies, and notices to the manufacturer, regarding either alterations to the hood or a new

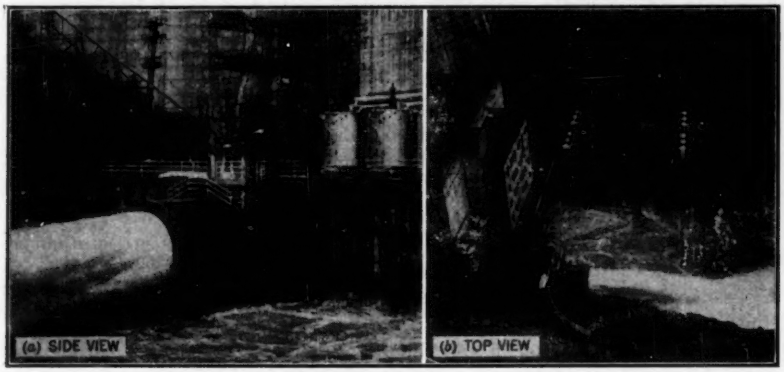

FIG. 18.—ALDER DAM (WASHINGTON) VALVE OPERATING UNDER FULL HEAD

type of hood, the writer determined, by crude means, that a cylinder with both ends open and one diameter in length beyond the point of impingement of the jet would transform the free-spreading jet into a cylindrical shape. The manufacturer expedited laboratory check tests while the fixed cylinder hood design was prepared and the steel liner was ordered.

Fig. 18 shows the new, 14-ft diameter, cylindrical hood at full discharge. Fig. 18(*b*) indicates the tailwater depressor effect of approximately 4 ft produced by the valve. The pronounced flattening or inward depressing of the jet on both sides, on the top, and on the bottom, which is believed to indicate self-venting of the interior of the jet, is shown in these photographs.

The movable hoods were abandoned because in tests of the Alder Dam valve, at full open position, the motor stalled in closing and the valve had to be closed by hand to a one-half opening before the motor could close it completely. After some investigation, from which it was noted that the flare of the jet was greater than that of the conical frustrum, observations with 3 pressure gages, tapped into the frustrum, disclosed an upstream component too great for the motor to overcome.

Air Demand.—The authors' statement (in the "Synopsis") that "*** the air demand is a function of the structure surrounding the valve ***" appears to be verified by experience and observation of the Alder Dam valve and others. In the first movable hoods at Alder and La Grande dams—the first such hoods ever used, in so far as is known—no air was admitted, either in the model which the manufacturer arranged to be made and tested at a government laboratory, or in the prototypes. Nevertheless, this combination of valve and hood operated with no sensible vibration. The observer's teeth could be placed against the flanges between sleeve and hood with very little unpleasant sensation.

Had the motor and operating gear been designed to overcome the upstream thrust safely, the pitting might have been obviated at slight maintenance expense by periodic welding or by the application of a welded circumferential facing of stainless steel approximately 12 in. wide. Sufficient air possibly to eliminate pitting might have been admitted at this point by the simple expedient of drilling a ring of closely spaced holes, provided the situation had been fully understood at the time. However, much air might have been required to satisfy the demand and the discharge coefficient might have been greatly reduced, or other difficulties resulted.

Negative pressures and cavitation undoubtedly occurred at the extreme upper end of the conical frustrum at certain gate openings, however, the available gages could not indicate them. Readings were taken at each one-tenth gate opening up to five-tenths, at which point the gages were wrecked internally by the rapid pressure variations. It was noted that the pressure curves plotted from these crude experiments, with the three gages tapped in along the top element of the frustrum, reversed themselves between the upstream and downstream gages.

For the 14-ft-diameter, fixed-cylinder, Alder Dam valve hood, an air inlet area of approximately 50 sq ft was provided above the upstream end of the cylinder, based on the manufacturer's model tests. At the smaller gate openings air rushed through the gratings at relatively high velocity. There was no back-lash of spray at these openings, and the slots cut in the conical jet by the horizontal and vertical diaphragms could be observed readily. It is probable that the air passed downstream through these slots. Part of it may also have passed through the aerated outer periphery of the jet itself.

Where there is no surrounding structure it is apparent that, were it not for impedence of the atmosphere, the conical jet would spread to infinity, with infinite diameter and, if conceivable, zero density. Actually, in this case, the air may be considered to be the surrounding structure, which impedes as well as aerates the jet, both from within and from without. Assuming that the thickness of the jet is constant, with a valve diameter of 66 in., expanding to the limit of the 14-ft-diameter hood, the density of the solid jet must be reduced in the ratio 66:168, or to approximately 40%, as it strikes the 14-ft-diameter hood. The percentage of water content is further reduced by the slight increase in the thickness of the expanding jet, which must obtain this air, as found by the authors.

As the Alder Dam valve opening increased from about one-half gate to full gate, the air drawn through the grill progressively decreased to zero, and the back-lash of spray increased considerably. However, this back-lash was not sufficient, as a water curtain, to cause the observed decrease in the air intake or demand. The upper part of the valve was still visible since the spray fell and was ejected by the bottom of the jet. This characteristic of the air demand appears to be similar to that indicated in Figs. 10 and 12.

From the experiences with the Alder Dam valve, the writer formed the conclusion that a hood of the Alder Dam type does not actually require any air upstream from the jet because the jet can procure the necessary air through the sides of the already aerated cylindrical jet. This is believed to be indicated by the depression in the sides and on the top and bottom of the jet (Figs. 18 and 19). The reason for the flattening of the jet, occurring at the top, the sides, and the bottom, rather than elsewhere, may be the influence of the "fins" of water which appear downstream of the diaphragms or vanes when the jet is not confined, as has been observed in some model tests.

It is suggested that, where the spreading jet is discharged into a tunnel, or into a chamber with baffles or dissipators—so that it may be difficult for the downstream air to balance the reduced pressures—the installation may act on the "ejector" principle. The valve then takes great quantities of upstream air, depending on the efficiency of the combination as an ejector, and that demand or need for upstream air may be eliminated, or greatly reduced, by omitting the baffles, unless these are an absolute necessity for dissipating the energy at that point. The Alder Dam installation contains no baffles, and the hollow jet is directed down the solid rock stream bed. Excess energy is absorbed by aeration and by water-cushioning in the small depression dug by the jet in the rock channel. There is very little solid rock into which such a jet will not dig to some extent.

Discharge Coefficients.—Difficulties similar to those described by the authors were encountered in gaging the discharge of the Alder Dam valve. Fig. 19 shows the discharge coefficients for the Alder Dam valve computed from the gaging results by Mr. Hickox, plotted on curves for the Chatuge, Fontana, and Nottely installations (Tennessee Valley Authority). The data were given to Mr. Hickox by H. Wiersema and Mr. Fry. The coefficient at the 50%-valve-opening was obtained from measurements taken with the original movable hood in place and conforms closely to the curve of all other coefficients that were obtained with the fixed hood, open at both ends. Apparently neither type of hood, nor the Fontana Dam dissipators, impaired the coefficient of discharge.

From Fig. 5, the lip of the end of the Alder Dam pipe (within the valve) was shaped like that at Fontana Dam, but if, as is apparent, the upstream position of the end of the valve sleeve at Fontana Dam was nearer the lip than the others, it invalidates the writer's previous impression that the "falling off" of the coefficients in the Chatuge and Notteley valves was caused by overtravel of the sleeves. In the Bonneville tests it was found that the maximum discharge of the models occurred at about 0.94 gate, as the control changed from

the end of the sleeve to the end of the pipe. The curves of the Alder and Fontana dams do not fall off at about 0.90 gate as do the others.

The limit switches at Alder Dam were set so that the indicator read exactly 0 and 1.00 in fully closed and fully opened positions, respectively. Not having the opportunity, subsequent to the Bonneville tests, to investigate this feature at Alder Dam, the writer assumed that the Alder Dam sleeve did

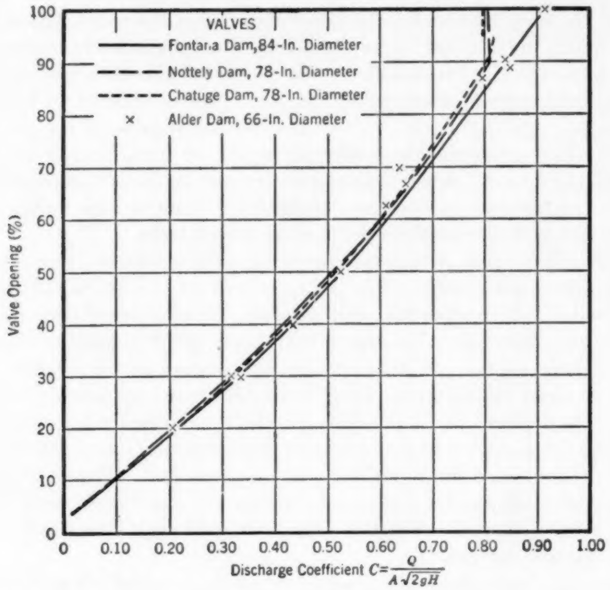

FIG. 19.—COMPARISON OF DISCHARGE COEFFICIENTS FOR SEVERAL FIXED-DISPERSION CONE VALVES

not overtravel or that the control change from sleeve to pipe. This appeared to be a proper assumption since there was no pitting of the ends of either sleeve or pipe after several years of operation, as might be expected from the negative pressures found in the Bonneville models under the end of the sleeve with the latter in the wide open position. There seems, therefore, as suggested by the authors, little point in having the sleeves overtravel, or the indicators register other than 0.0 and 1.00 in closed and open positions.

Vibration and Noise.—There is no sensible vibration at any gate opening of the Alder Dam valve or hood, and no loud noise at any critical opening, such as is described by the authors. There is a slight "wire drawing" sound at the last "pinch off" in seating the valve in closed position, which is neither annoying nor detrimental. It is conjectured whether the "howl" mentioned by the authors is a matter of acoustics and reverberation, rather than harmful vibration. In the silent Alder Dam powerhouse, a slight acoustic effect from the valve initially caused some head shaking of experienced operators and en-

gineers. However, this was quickly forgotten and overshadowed by the much higher noise levels caused by the generators when the latter were in operation.

Necessity for Energy Absorption and River-Bed Protection.—Where design conditions require that valves be located in tunnels or chambers, the problems of directing the jets; supplying sufficient air; and, possibly, dissipating some of the energy are recognized. Such requirements evidently introduce many variations in results such as back-lash of water, noise, and other considerations. It seems desirable, if possible, to place the valve at the downstream end of the conduit with a slight reducer section, and to impede its discharge as little as possible, directing it so that it will not harm adjacent structures. The valves at Alder and La Grande dams have dug moderate holes or grooves in the rock river bed forming their own water cushions, at no cost for excavations or concrete lining, and small cost for clearing away the resulting debris. The aerated cylindrical jets have much less impact and digging power than the solid jets of needle valves. At Alder Dam the toe of the rock fill of the service road, although not more than 15 ft away from the point where the jet strikes the tailrace, has not been undermined in about eight years of operation of the valve.

Suggested Trend in Future Design.—Experiments with jets impinging upon plates at various angles have indicated that there is a variable tendency toward reverse flow which decreases as the angle of incidence decreases. This may be the basic cause of back-lash in the Alder Dam hood and, in general, wherever a spreading jet is confined. This condition seems to indicate an advantage in constructing the valves with "splitter" cones of longer taper.

The writer concurs with the authors' suggestion as to care in the placement and secure fastening of the appurtenances. At Alder Dam the spray caused the frail nipples supporting the two relatively heavy grease cups on the operating screws to break off. The cups were found dangling in the spray by the small copper grease lines, which themselves did not break loose at the upper end. The grease lines were removed and separate alemite fittings were installed. Certain other bolts and fastenings of the ladders also became loose and were welded. There is no loud noise, and the spray is of insufficient force to remove paint from the shafts or grease from the exposed operating screws or large bronze sleeve.

The differences in the behavior of the several installations suggest the advantage of studies toward the development of a separate chamber or discharge outlet for each valve, and toward the elimination of all possible baffles or obstructions that would impede air supply from downstream, thereby reducing upstream air demand. An attempt should be made to develop a light, movable, vented or unvented hood similar to the original hoods at Alder and La Grande dams, with the exception that the hoods and water passages be streamlined, and that the motors and operating gear be designed for those conditions. Having all moving parts exposed would facilitate inspection and maintenance, unhampered by back-lash of spray.

REX A. ELDER,[10] M. ASCE, and GALE B. DOUGHERTY,[11] A.M. ASCE.—The reception given this paper by the discussers and the diversity of phases of the subject discussed have been highly gratifying. Each discussion has added to the total value of the paper and thus to the fund of engineering knowledge.

Mr. Murphy wrote in reference to the air-demand results that "*** These finding were based on a particular case and, unless all features are duplicated, the same results cannot be expected in other cases. ***" The writers heartily endorse this view and feel it should be noted by all who wish to use the data.

The TVA hydraulic laboratory has never made a model study of the operating characteristics of the valve. The writers therefore cannot make a model-prototype comparison. The writers agree with Mr. Murphy that such a comparison could be of great value. They hoped that others might have made such studies.

Mr. Ballester has inquired about changing the central angle of the deflector cone and the angle between the axis of the valve and the conduit in order to change the size and shape of the jet. The authors know of no data on changes in the central angle of the cone. However, recent (1952) studies at the TVA hydraulic laboratory on proposed changes in valve location at the Nottely and Chatuge projects might be of interest with respect to valve location. At these projects, the valves are to be connected to the turbine scroll case as shown in Fig. 20. In this illustration, the heavy broken line indicates the limits of the jet impact area. The tailwater elevation is 1612 and the center line of the valve is at El. 1613.5. In studying the jet action in the model, it was found that the jet pattern was unsymmetrical, having heavier flow on the right side (Fig. 20).

Mr. Ballester and Mr. Siao each used different areas for determining the discharge coefficient. The writers used the net area through the body of the valve—not the cylindrical opening as Mr. Ballester understood. This measurement was selected in preference to the others because (1) it provided the simplest and easiest approach for a designer concerned with determining the proper valve size and (2) it provided the simplest computation approach for preparing a table of discharges for various gate openings. In response to Mr. Ballester's suggestion, however, the discharge coefficients for one test each from the Nottely and Fontana data have been computed using his definition of area. The results are shown in Fig. 21 and compared with Mr. Ballester's results.

Mr. Siao's approach is very interesting, and within the limits he defined could probably be used by the maunfacturer's designers in studying the effect of changes in their cone dimensions.

Mr. Siao calls attention to the shape of the issuing jet shown in Fig. 13. This sketch was intended to show not basic shapes but merely general nomenclature. The writers agree with Mr. Siao that the jet shape probably is not exactly as shown, but they do not have any data.

[10] Head, Hydr. Lab. Sections, TVA, Norris, Tenn.

[11] Hydr. Engr., TVA Hydr. Lab., Norris, Tenn.

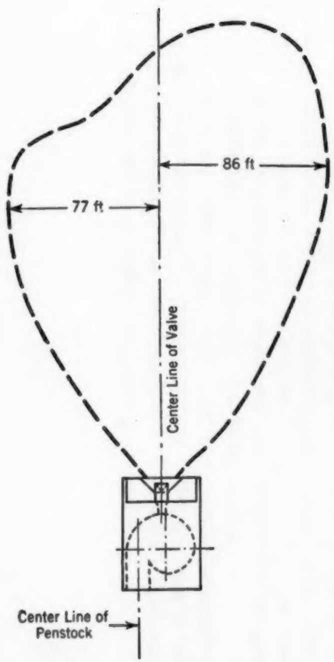

FIG. 20.—JET PATTERN OF A MODEL

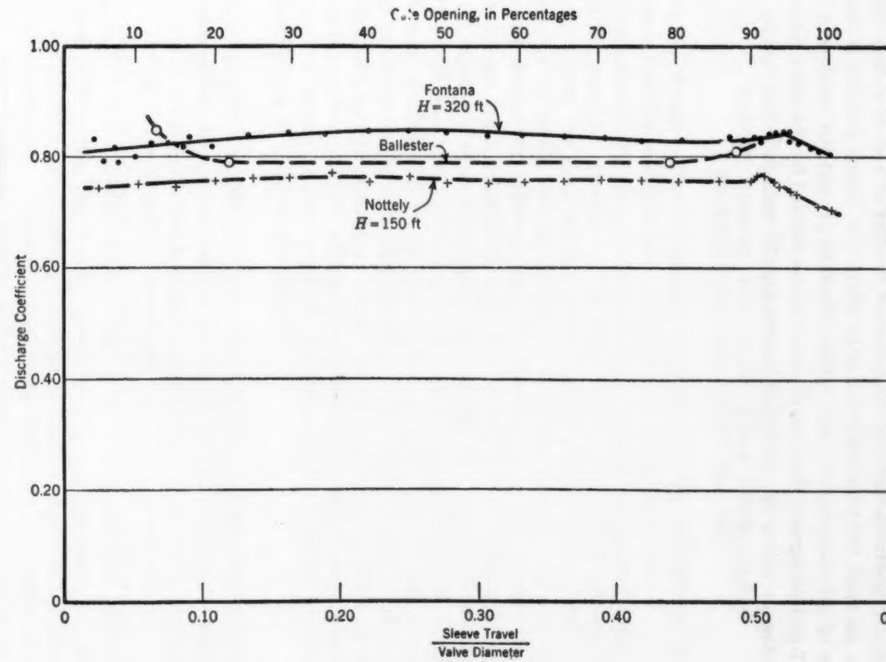

FIG. 21.—DISCHARGE COEFFICIENTS

Mr. Gongwer's observations at Alder Dam are interesting and informative. When sleeve positions based on an indicator reading of 1.00 at a fully opened position are used, the question arises as to where that actual position is located. Because of this uncertainity, the writers chose to present the coefficient data in Fig. 7 plotted against the ratio of sleeve travel divided by the valve diameter. If homologous valves are used, this system should yield definable results.

DISCUSSION OF
RATING CURVES FOR FLOW OVER DRUM GATES
PROCEEDINGS-SEPARATE NO. 169

GUIDO WYSS[6].—The information presented by Mr. Bradley is of utmost value for determining the quantities of discharge over drum gates under various heads for any gate position. This information will permit operators in the field to adjust the gate position from corresponding chart values in such a manner as to obtain the desired flow. The use of drum gates as an actual metering device for spillway quantity discharges is unique and the results obtained are more practicable and reliable than those obtained by stream gaging, especially when this gaging is conducted during periods of high floods.

[6] Mech. Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

It would have been interesting if the author had presented an investigation of the flow, profiles of the upper and lower nappe surfaces, as well as the actual water pressures on the upstream plate of the drum gate by use of charts. This would afford an opportunity to obtain the true loading conditions on the gate during the cycle of operation from fully-raised gate to fully-lowered gate. This information would be important in the determination of the buoyancy and loading criteria of the gate structure.

SAM SHULITS,[7] M. ASCE.—An outstanding contribution to the design and operation of drum gates has been presented in this report of the author's work at the USBR. The paper and its complement[2] fill a great need.

[7] Associate Prof., Director, Hydr. Lab., Civ. Eng. Dept., Pennsylvania State College, State College, Pa.

[2] "Discharge Coefficients for Irregular Overfall Spillway Sections," by J. N. Bradley, *Engineering Monograph No. 9*, Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo., March, 1952.

Since 1928, when the Freeman Scholarships were established, there has been a tremendous development of hydraulic model research in the laboratories of the United States. Although these laboratories are unexcelled in size and quality, many hydraulic engineers have pondered the procession of models (spillways, stilling pools, and river reaches) in the period from 1928 to 1953 with few, if any, summaries or proposals for design to reduce the dependence on models. In Mr. Bradley's work there is strong evidence that the laboratories will produce correlations and syntheses—not more models.

When it is realized that many of the most famous and productive laboratories in the United States did not exist prior to 1928, the lack of correlation and synthesis for general use is understandable. The hope is that other works of similar quality will be added to engineering literature.

Bob Buehler,[8] A. M. ASCE.—An interesting and clever use of data has

[8] Hydr. Engr., TVA, Knoxville, Tenn.

resulted in a method by which records of gate settings at dams can be made a substitute for missing stream-flow records and can be used to augment existing records. The construction of a dam and reservoir often floods an established stream gage. Unless the gage is replaced below the dam or upstream from the reservoir, subsequent stream flow usually is not accurately known. Sometimes a series of dams (each causing the water to back up to the dam above) prevents continuing established gages at the strategic points where they had been located. The less accurate—and more costly—slope stations are not completely satisfactory alternatives to the single-line rating stations.

If the spillway of a dam can be rated with an accuracy comparable to the accuracy obtainable with a gage (as demonstrated by Mr. Bradley for certain spillway types), and if allowance is made for flow through other water outlets such as turbines, locks, and sluices, the structure is then superior in some respects to the gage. For example, the rating of the dam should be permanent, whereas the rating of a gage usually requires frequent checking.

Mr. Bradley's method for rating drum gates not only allows records for ordinary stream flow to be supplemented, but also probably gives a more accurate determination of extreme flood rates than do most gages. He has made an important contribution to the planning and design of drum-gated structures.

The author has presented a method for rating a spillway at all heads provided the coefficient for one appreciable head is known. He also states that a coefficient for the designed head can be estimated for most spillways by a method previously published.[2] The writer, on the other hand, offers a method by which an ogee spillway can be rated, provided its profile shape is known. The method is based on an equation derived by R. N. Brudenell, A. M. ASCE, incidental to studies made on radial gates.[9] Mr. Brudenell's equation is

[9] "Flow over Rounded Crests," by R. N. Brudenell, *Engineering News-Record*, July 18, 1935, p. 95.

$$Q = \frac{3.97\, L\, H^{1.62}}{H^{0.12}{}_{D}} \qquad (1)$$

in which Q is the spillway discharge, in cubic feet per second; L denotes the length of the spillway, in feet; H is the total head on the spillway crest, in feet; and H_D represents the design head in feet. The design head is that head which produces a standard lower nappe that agrees closely with the spillway profile.

Eq. 1 was intended to be used with heads greater than $H_D/4$, although the equation has been found to agree closely with model data for somewhat lower heads. Without knowing any coefficients, Eq. 1 gives discharges that agree closely with those obtained by Mr. Bradley for Black Canyon Dam. In the case of Black Canyon Dam, Mr. Bradley used one known coefficient and the curve of Fig. 7. Free-flow discharges computed by the two methods are shown in Cols. 2 and 3, Table 5. The procedure by which Eq. 1 was applied will be described subsequently.

It is assumed that in choosing Black Canyon Dam for his example, the author knew that his method would yield discharges close to known values. The good agreement for all except the low heads shows that, in this example, Eq. 1 (using only the shape of the spillway) also produces suitable results. This good agreement suggests, too, that there must be a close relationship between the curve in Fig. 7 and a similar curve that can be derived from Eq. 1. To examine the relationship, theoretical discharge coefficients were computed by using

$$Q = C_q \, L \, H^{3/2} \qquad (2)$$

and Eq. 1, from which

$$C_q = \frac{3.97 \, H^{1.62}}{H^{0.12}_D \, H^{3/2}} \qquad (3)$$

TABLE 5.—FREE DISCHARGES FOR BLACK CANYON DAM IN IDAHO

Total head, in feet	Discharge, in cubic feet per second[a]	Using Eq. 1		Using Fig. 14	
		Discharge, in cubic feet per second	Difference, in percent	Discharge, in cubic feet per second	Difference, in percent
(1)	(2)	(3)	(4)	(5)	(6)
17	15,950	15,847	−0.65	15,910	−0.25
16	14,420	1,4363	−0.39	14,421	−0.01
14.5[b]	12,296	12,247[c]	−0.40	12,296	0
12	9,072	9,013	−0.65	9,049	−0.25
10	6,759	6,708	−0.75	6,735	−0.36
8	4,736	4,673	−1.33	4,692	−0.93
6	2,949	2,932	−0.58	2,944	−0.20
4	1,514	1,521	+0.46	1,527	+0.86
3	943	954	+1.17	958	+1.59
2	478	494	+3.35	496	+3.76

[a] From Col. 6, Table 3. [b] Head at which $C_q = 3.48$. [c] C_q would be 3.466 for this discharge.

The design head, H_D, was found (by a method to be described subsequently) to be 45 ft for Black Canyon Dam, and this value was used in making the test. Thus, for $H_D = 45$ ft,

$$C_q = \frac{2.5143 \, H^{1.62}}{H^{3/2}} \qquad (4)$$

For several assumed values of total head, H, varying from 2 ft to 58.5 ft, corresponding C_q-values were computed. The resulting C_q of 3.97 for a head of 45 ft (H_o) was taken arbitrarily as the known coefficient, C_o. Then the (H/H_o)-ratios and the (C_q/C_o)-ratios were computed for all other heads in the assumed range. The resulting curve is the solid line in Fig. 14. The dashed curve is from Fig. 7. The agreement is close—as expected. Still using H_D equal to 45 ft, the remainder of the process was repeated using the coefficient for the 25-ft head as C_o, and then using the coefficient for the 12-ft head as C_o. There was no discernible difference in the curves resulting from the three separate selections. A similar procedure, using H_D equal to 20 ft in Eq. 1, also showed no differences from Fig. 14. It can probably be proved that there should be no difference.

The curve derived from Eq. 1 then was applied to the Black Canyon Dam spillway, assuming (as did the author) that the coefficient is 3.48 at a 14.5-ft head. The resultant free discharges are shown in Col. 5, Table 5.

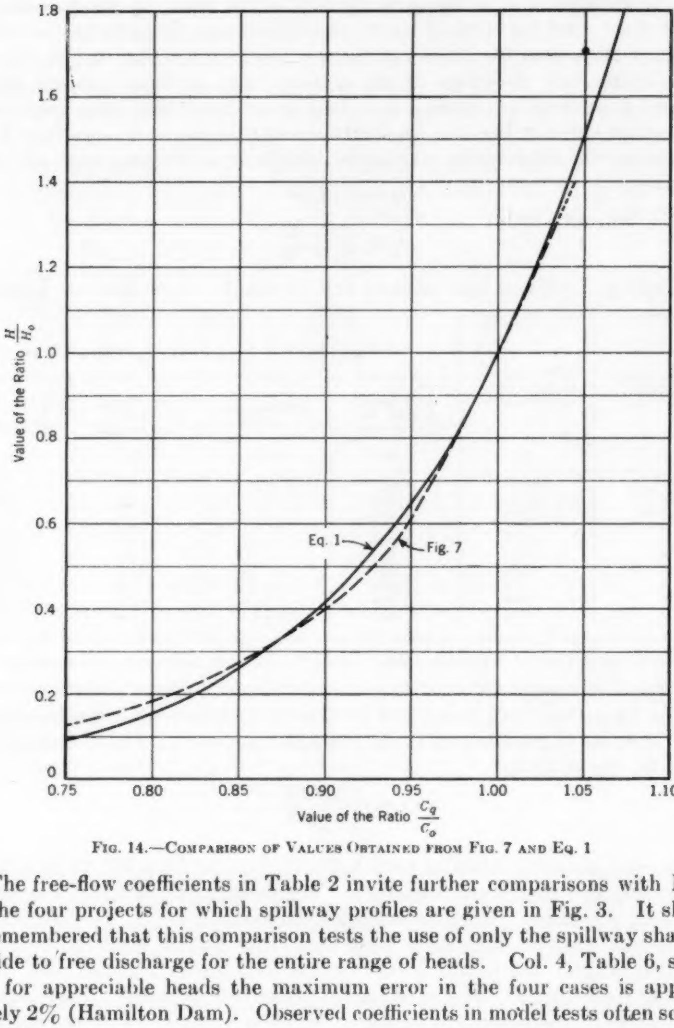

FIG. 14.—COMPARISON OF VALUES OBTAINED FROM FIG. 7 AND EQ. 1

The free-flow coefficients in Table 2 invite further comparisons with Eq. 1 for the four projects for which spillway profiles are given in Fig. 3. It should be remembered that this comparison tests the use of only the spillway shape as a guide to free discharge for the entire range of heads. Col. 4, Table 6, shows that for appreciable heads the maximum error in the four cases is approximately 2% (Hamilton Dam). Observed coefficients in model tests often scatter as much.

The same coefficients permit testing the curve in Fig. 7 for all eleven spillways. This test is not as severe, however, because it is necessary to assume one known coefficient at which head agreement becomes perfect. At near-by higher and lower heads, large divergences would not be expected. Col. 6, Table 6, shows that for appreciable heads the maximum error is slightly greater than 2% (Hoover Dam, shape 8-M5). The base coefficient selected to obtain C_q from the (C_q/C_o)-ratios is designated by a footnote for each project. These arbitrary selections were made for medium high heads.

TABLE 6.—Comparison of Free-Flow Spillway Coefficients

Total head, in feet	Coefficient obtained from model test	Using Eq. 1		Using Fig. 7		Using Fig. 14	
		C_q	Difference, in percent	C_q	Difference, in percent	C_q	Difference, in percent
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Grand Coulee Dam							
35	3.920			3.914	− 0.15	3.902	−0.46
30	3.842			3.831	− 0.29	3.827	−0.39
25	3.745			3.745[a]	0	3.745[a]	0
20	3.635			3.655	+ 0.55	3.651	+0.44
15	3.510			3.550	+ 1.14	3.524	+0.40
10	3.352			3.370	+ 0.54	3.356	+0.12
5	3.220			3.138	− 2.54	3.168	−1.62
Bhakra Dam							
28	3.680			3.736	+ 1.52	3.732	+1.41
23	3.645			3.645[a]	0	3.645[a]	0
18	3.550			3.547	− 0.08	3.543	−0.20
13	3.420			3.434	+ 0.41	3.404	−0.47
8	3.275			3.215	− 1.83	3.208	−2.04
3	3.120			2.748	−11.92	2.854	−8.53
Shasta Dam							
38	3.895			3.910	+ 0.39	3.899	+0.10
33	3.835			3.839	+ 0.10	3.831	−0.10
28	3.760			3.760[a]	0	3.760[a]	0
23	3.675			3.677	+ 0.05	3.674	−0.03
18	3.575			3.591	+ 0.45	3.568	−0.20
13	3.465			3.455	− 0.29	3.429	−1.04
8	3.335			3.215	− 3.60	3.230	−3.15
Hamilton Dam, H_D = 52 Ft							
35	3.710	3.785	+2.02	3.741	+ 0.84	3.730	+0.54
30	3.645	3.716	+1.95	3.662	+ 0.47	3.659	+0.38
25	3.580	3.635	+1.54	3.580[a]	0	3.580[a]	0
20	3.500	3.539	+1.11	3.494	− 0.17	3.490	−0.29
15	3.400	3.420	+0.59	3.394	− 0.18	3.369	−0.91
10	3.290	3.258	−0.97	3.222	− 2.07	3.208	−2.50
5	3.160	2.997	−5.16	3.000	− 5.06	3.029	−4.14
Friant Dam							
20	3.650			3.717	+ 1.84	3.706	+1.53
17	3.625			3.639	+ 0.39	3.632	+0.19
14	3.550			3.550[a]	0	3.550[a]	0
11	3.460			3.458	− 0.06	3.452	−0.23
8	3.340			3.348	+ 0.24	3.319	−0.63
5	3.175			3.142	− 1.04	3.131	−1.38
2	2.965			2.723	− 8.15	2.812	−5.16

[a] Coefficient assumed to be known.

TABLE 6.—(*Continued*)

Total head, in feet	Coefficient obtained from model test	Using Eq. 1		Using Fig. 7		Using Fig. 14	
		C_q	Difference, in percent	C_q	Difference, in percent	C_q	Difference, in percent
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Norris Dam, H_D = 35 Ft							
35	3.915	3.969	+1.38	3.934	+ 0.49	3.923	+0.20
30	3.845	3.897	+1.35	3.852	+ 0.18	3.848	+0.08
25	3.765	3.812	+1.25	3.765[a]	0	3.765[a]	0
20	3.670	3.711	+1.12	3.675	+ 0.14	3.671	+0.03
15	3.550	3.586	+1.01	3.569	+ 0.53	3.543	−0.20
10	3.390	3.416	+0.77	3.388	− 0.06	3.373	−0.50
5	3.125	3.143	+0.58	3.155	+ 0.96	3.185	+1.92
Madden Dam							
35	3.900			3.825	− 1.92	3.814	−2.20
30	3.770			3.744	− 0.69	3.740	−0.80
25	3.660			3.660[a]	0	3.660[a]	0
20	3.560			3.572	+ 0.34	3.568	+0.22
15	3.460			3.470	+ 0.29	3.444	−0.46
10	3.365			3.294	− 2.11	3.279	−2.55
5	3.280			3.067	− 6.49	3.096	−5.61
Capilano Dam, H_D = 48 Ft							
33	3.775	3.797	+0.58	3.783	+ 0.21	3.775	0
28	3.705	3.720	+0.40	3.705[a]	0	3.705[a]	0
23	3.625	3.634	+0.25	3.623	− 0.05	3.620	−0.14
18	3.530	3.529	−0.03	3.538	+ 0.23	3.516	−0.40
13	3.415	3.394	−0.62	3.405	− 0.29	3.379	−1.05
8	3.250	3.201	−1.51	3.168	− 2.52	3.183	−2.06
Hoover Dam Shape 4-M3, H_D = 50 Ft							
26	3.670	3.670	0	3.681	+ 0.30	3.677	+0.19
22	3.605	3.597	−0.22	3.605[a]	0	3.605[a]	0
18	3.540	3.512	−0.79	3.526	− 0.40	3.522	−0.51
14	3.472	3.408	−1.84	3.439	− 0.95	3.414	−1.67
10	3.405	3.273	−3.88	3.306	− 2.91	3.280	−3.67
6	3.338	3.077	−7.82	3.064	− 8.21	3.082	−7.67
Hoover Dam Shape 8-M5							
28	3.735			3.814	+ 2.12	3.800	+1.74
25	3.705			3.752	+ 1.27	3.749	+1.19
20	3.650			3.650[a]	0	3.650[a]	0
15	3.565			3.537	− 0.78	3.530	−0.98
10	3.460			3.387	− 2.11	3.358	−2.94
5	3.335			3.059	− 8.28	3.088	−7.41
Hoover Dam Shape 7-C4							
26	3.665			3.691	+ 0.71	3.687	+0.60
22	3.615			3.615[a]	0	3.615[a]	0
18	3.540			3.535	− 0.14	3.532	−0.23
14	3.450			3.449	− 0.03	3.423	−0.78
10	3.360			3.315	− 1.34	3.290	−2.08
6	3.200			3.073	− 3.97	3.091	−3.41

The solid-line curve in Fig. 14 also was tested in this manner. The same coefficient at each project was assumed to be known as when the curve in Fig. 7 was tested. Col. 8, Table 6, shows that for appreciable heads the maximum error is slightly more than 2% (Madden Dam).

These comparisons show that the direct application of Eq. 1, Fig. 7 (or Fig. 14) (derived from Eq. 1), all give highly accurate free-flow spillway discharges for ogee dams at all but low heads. Eq. 1, applied directly to the spillway shape, has the advantage that no coefficients need be known or estimated in advance.

The comparisons in Table 6 show a tendency toward errors of some importance at low heads when Eq. 1 or its companion curve in Fig. 14 is used, as well as when Fig. 7 is used. In most cases the errors are negative. These errors are of little concern in planning the safety of a structure against extreme floods, or in considering most other operations such as emptying the reservoir. The errors nonetheless affect the analytical rating of drum gates in the lowered or slightly raised positions. The free-flow coefficients help to determine the direction of the general curves at the large negative angles shown in Fig. 6. Free discharges form the base curve of the rating curves in Fig. 12 and help define the curvature of the low ends of the cross-plot curves in Fig. 13. Low to ordinary heads, corresponding to normal stream flow, can exist for a large part of the time at dams whose reservoir capacities are small. Further study of data for low heads might lead to valuable refinements.

Application of Eq. 1.—Since the factor H_D in Eq. 1 represents the head at which a standard lower nappe shape is a reasonable approximation of the spillway shape (as designed or built), it is only necessary to find this head to apply the formula. Spillway coordinates for a standard crest having a vertical upstream face have been used to find this head.[10] These coordinates are shown in Table 7. The last column in Table 7 refers the horizontal (x) coordinates to the spillway crest because this form is the simplest to apply. In Table 7, y is the distance below the crest elevation.

[10] "Hydroelectric Handbook," by William P. Creager and Joel D. Justin, John Wiley & Sons, Inc., New York, N. Y., 2d Ed., 1950, p. 362.

TABLE 7.—COORDINATES OF A STANDARD SPILLWAY CREST

Value of $\frac{x}{H_D}$	Value of $\frac{y}{H_D}$	Value of $\frac{x}{H_D}$ referred to crest
0	0.126	−0.3
0.1	0.036	−0.2
0.2	0.007	−0.1
0.3	0	0
0.4	0.007	0.1
0.6	0.063	0.3
0.8	0.153	0.5
1.0	0.267	0.7
1.2	0.410	0.9
1.4	0.590	1.1
1.7	0.920	1.4
2.0	1.31	1.7

Using these dimensionless coordinates, standard spillway shapes were plotted (Fig. 15) for values of H_D from 10 ft to 60 ft. In Fig. 15 negative horizontal distances indicate the distance upstream from the crest. The spillway shape as designed or built is then drawn on transparent paper. This paper is laid over Fig. 15 and the value of H_D which gives the best fit is selected. In deciding the best fit it may be found that the profile upstream from the crest indicates one value and the downstream profile indicates a different value.

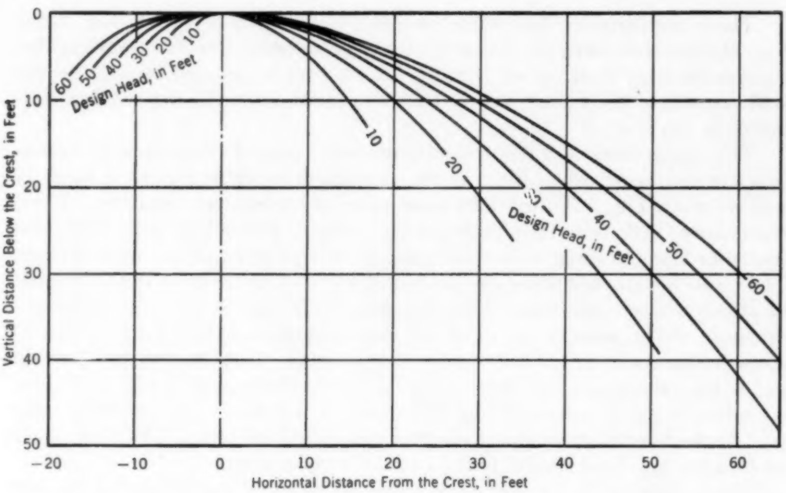

Fig. 15.—Standard Spillway Shapes

The higher of the two indicated values of H_D should be used. For example, the shape of Black Canyon Dam spillway upstream from the crest indicated a value of approximately 45 ft for H_D. The downstream shape indicated a value of approximately 25 ft. The larger value was used.

The determination of the H_D-value which gives a reasonable fit requires a certain amount of judgment. When the profile upstream from the crest is the criterion, the lip of the dam will sometimes be the determinant. Sometimes, however, the lip droops sharply downward and indicates a lower value than other parts of the upstream profile. When the downstream shape is the criterion, good results have been obtained by assigning a value of H_D based on the average fit in the zone between points on the spillway where tangents range from 20° to 35° from the horizontal. The exact value of H_D is not too important. Since it enters Eq. 1 in the 0.12 power, a difference of 10% in its value affects the discharge by only 1.15%.

The writer's application of Eq. 1 has been limited to fairly high dams. Although the total head used in Eq. 1 should include the approach velocity, the accuracy of Eq. 1 when used for low dams, where approach velocity is large, has not been tested.

So far as is known, the application of standard nappe shapes (for which discharge coefficients are known) to actual spillways on a basis of reasonable best fit was first suggested by W. M. Borlund.[11] Mr. Borlund used a curve of observed C_q-value plotted against H/H_o. In 1942, C. E. Kindsvater, M. ASCE, suggested a similar procedure in which the curve of C_q versus H/H_o was derived from Eq. 1. Mr. Kindsvater's work (not published) should give results comparable to those obtained herein.

The material presented is regarded as an excellent check on that part of Mr. Bradley's work which relates to free discharge over an ogee spillway.

[11] "Flow over Rounded Crest Weirs," by W. M. Borlund, thesis presented to the University of Colorado, at Boulder, Colo., in 1938, in partial fulfilment of the requirement for the degree of Master of Science.

F. B. CAMPBELL,[12] M. ASCE, AND A. A. McCOOL,[13] J. M. ASCE.—The

[12] Chf. Hydr. Engr., Analysis Branch, Corps of Engrs., U. S. Waterways Experiment Station, Vicksburg, Miss.

[13] Hydr. Engr., U. S. Waterways Experiment Station, Vicksburg, Miss.

experimental data on discharge coefficients for flow over drum gates are a welcome addition to the published information on flow over spillways, or the observation and recording of the flow of streams. A paper by Robert E. Horton has been a guide for the estimation of flows over spillways since its publication.[14]

[14] "Weir Experiments, Coefficients and Formulas," by Robert E. Horton, *Water Supply and Irrigation Paper No. 200*, U. S. G. S., U. S. Gov't Printing Office, Washington, D. C., 1907 (Revision of Paper No. 150).

The basic information for discharge over curved crests which fit the under side of a nappe from a sharp-crested weir can be deduced from investigations made by Bazin,[15, 16] although the published record of these experiments has not been

[15] "Recent Experiments on the Flow of Water over Weirs," by M. Bazin, *Annales des Ponts et Chaussées*, October, 1888 (Translation by Arthur Marichal and John C. Trautwine, Jr.), *Proceedings*, Engineers' Club of Philadelphia, Pa., Vol. VII, No. 5, 1890, p. 259.

[16] *Ibid.*, Vol. IX, No. 3, 1892, p. 231.

generally available to engineers in the United States. The investigations conducted by the USBR (proposed by E. W. Lane, M. ASCE) embraced and extended the scope of Bazin's work which is often used as the basis for overflow spillway shapes.[3] Although good estimates for discharge over free-overflow

[3] "Studies of Crests for Overfall Dams," by E. W. Lane, *Bulletin No. 3*, Part VI, Boulder Canyon Final Reports, Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo., 1948.

crests can be accomplished rather simply, the problem becomes complicated when flow through partly opened crest gates is involved.

The commonly used types of crest gates are vertical lift gates, tainter or radial gates, and drum gates. The coefficient for a partly opened vertical lift gate depends on the location of the plane of the skin plate or lip with respect to the axis of the curved crest. The discharge coefficient for tainter gates is affected by the radius of the skin plate, the elevation of the trunnion with respect to the crest, and the location of the gate seat with respect to the axis as well as the crest curvature. To complicate any investigations further, observers define the gate opening variously as (1) the length of the arch from the gate seat to the gate lip, (2) the vertical distance from the lip to the face, and (3) the distance from the lip to the face measured normal to the face. The last method is believed to give the proper dimension, whereas the foregoing considerations are geometrical. The effective head for a partly opened vertical lift or tainter gate depends on the pressures on the face of the concrete and the pressures within the issuing jet. Mr. Bradley has outlined the geometrical variables and the method of measuring for the analysis of partly raised drum gates.

To operate the drum gate requires no mechanical hoisting equipment—an excellent design feature. Many of the dams constructed by the USBR have spillways controlled by drum gates. For example the Arrowrock Dam in Idaho (constructed in 1915) and the (Tieton) Dam in Washington (constructed in 1925) are both equipped with drum gates. B. F. Thomas and D. A. Watt credit H. M. Crittenden with the design of what is apparently the first drum gate.[17] The gates were installed in Dam No. 1 on the Osage River in Missouri

[17] "The Improvement of Rivers," by B. F. Thomas and D. A. Watt, John Wiley & Sons, Inc., New York, N. Y., 2d Ed., 1913.

in 1911. However, the USBR has made refinements on earlier drum-gate operation.

The discharge coefficients presented by the author are based on model studies. There should be opportunity to check the coefficients for relatively low heads with partly raised gates in the prototype by current-meter measurements. Only on rare occasions with large floods is it possible to verify the coefficients for high prototype heads over the drum gates in the lowered position. The author's mention of the failure to obtain discharge measurements during the 1948 flood over the Grand Coulee Dam spillway emphasizes the importance of this condition. The writers have studied the basic data for high heads over the drum gate in the lowered position.

It becomes evident from a study of Table 2 that the ratio of gate radius to maximum head has a wide range. The writers use the ratio r/H_D, in which H_D is the design head for the spillway. This is the inverse of the ratio used by Mr. Bradley, used so that circular arcs can be traced on dimensionless profiles of X/H_D and y/H_D.

A comparison has been made of the coefficients for various (r/H_D)-values with the gate down. Only the high-overflow sections with negligible velocity of approach were selected from Table 2 for a study of discharge coefficients. Table 8 shows the value of the discharge coefficients for the condition when the drum gate is down. The percentage difference of the coefficient from that of the Madden Dam coefficient is also shown. It is expected that the accuracy of the discharge measurements and thus the coefficient of discharge is less than 1%.

TABLE 8.—COMPARISON OF DISCHARGE COEFFICIENT WITH THE GATE DOWN

Dam	Radius of gate, in feet[a]	Maximum head on crest, in feet[a]	Ratio, $\frac{r}{H_D}$	Coefficient,[b] C_q	Difference, in percent, from Madden Dam
Madden (Canal Zone)	30.0	30.0	1.00	3.77	0.0
Norris (Tennessee)	34.0	27.0	1.26	3.80	0.8
Grand Coulee (Washington)	66.2	31.6	2.09	3.87	2.6
Shasta (California)	66.2	28.0	2.37	3.76	−0.3
Friant (California)	47.0	19.0	2.47	3.64	−3.5
Capilano (British Columbia)	71.0	23.0	3.08	3.62	−4.0

[a] From Table 1. [b] From Table 2.

The dams for which the data are listed in Table 8 are in the approximate chronological order of the time of their design conception.

Because of the increase in the ratio of r/H_D (Table 8), it is of interest to plot the profile for the lower surface of the nappe from a sharp-crested weir with an approach slope of 2 on 3 in terms of X/H_D and J/H_D and to superimpose on it the arcs of circles with radii of r/H_D equal to 1, 2, and 3, as is done in Fig. 16. The center of the radius is located on the axis of the crest.

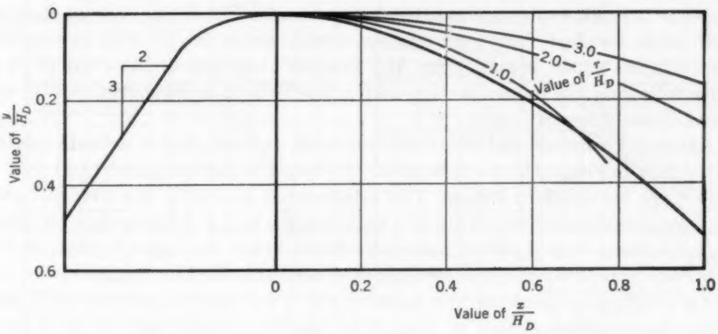

FIG. 16.—LOWER SURFACE OF NAPPE FROM SLOPING WEIR COMPARED WITH CIRCULAR ARCS

It can be seen that the arc represented by r/H_D equal to 1 is a fair approximation of the true nappe shape. The arcs of r/H_D equal to 2 and 3 indicate a very flat curvature in comparison to the shape of the nappe.

It might be assumed that for a crest with r/H_D equal to 3, C_q would be one third of that for the design head of a crest with r/H_D equal to 1. Model studies for Madden Dam reported by Richard R. Randolph, Jr.,[18] indicate that the coefficient for such a condition is approximately 3.40. Such a coefficient is not in agreement with that for Capilano Dam with r/H_D equal to 3.62 at full head. The lack of agreement does not necessarily vitiate the initial assumption. The difference in the coefficient may be caused by the difference in shape of the two crests upstream from the circular arc. Furthermore, the scale ratio of the Madden Dam model was only 1:78, and a 10-ft prototype head would be 0.128 ft on the model, which is near the lower limit of reliability for conformity of the discharge coefficient.

[18] "Hydraulic Tests on the Spillway of the Madden Dam," by Richard R. Randolph, Jr., *Transactions*, ASCE, Vol. 103, 1938, p. 1091.

Joseph N. Bradley,[19] A.M. ASCE.—Mr. Shulits' statements regarding

[19] Hydr. Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

the lack of correlation in laboratory studies are well founded, and the writer is in complete agreement with his views.

Mr. Buehler's analysis for the determination of the designed head, H_D, for overflow sections formed by a single radius, or for a shape that conforms closely to a single radius, gives satisfactory results. The comparison of discharge coefficients for free flow over various dams, using Eq. 3 with the method offered in the paper, is gratifying. Mr. Buehler's method certainly has merit because following the determination of H_D, coefficients of discharge can be computed directly for all heads.

Messrs. Campbell and McCool undertook to show that a definite relationship exists between the coefficient of discharge at the designed head and the ratio r/H_D for overflow shapes. This relationship is valid if the overflow shape can be approximated by an arc of a single radius and if the approach conditions are favorable—that is, if the approach depth below the crest is at least twice H_D. This method results in a coefficient of discharge for the designed head only. When overflow sections are encountered where a single radius does not approximate the overflow shape, or when the approach conditions are unusual, an engineering monograph[2] may prove helpful.

Mr. Wyss suggested that pressures and water surfaces for drum gates at various positions and reservoir levels would be useful to designers in computing gate loadings. A limited amount of information is available, and this will be presented.

Because there was good correlation among the discharge coefficients, it was reasoned that the pressures and related flow patterns would also correlate through the same variables.

Pressures and water-surface profiles are shown plotted in dimensionless coordinates (in terms of the radius of the gate) in Fig. 17. Five positions of the gate are shown for various reservoir levels producing flow over the gate. Pressures and water surfaces are shown for some levels whereas only pressures are available for others. The broken lines represent pressure, measured vertically, for the reservoir levels indicated at the left of the charts. Upper water-surface profiles are shown by solid lines, and lower water-surface profiles are identified by dash lines. The charts represent a composite, in graphical form, of information from model tests performed on the Grand Coulee, Hamilton, Norris, Friant, and Hoover dams.

To determine graphically the most adverse water load on a particular gate, it is necessary to investigate the pressures for several gate positions. Assuming that the first position is $\theta = 41°$, the gate is drawn in this position on a piece of transparent paper to the same scale as that used in Fig. 17. The maximum expected reservoir is indicated for this gate position on the left side of the transparent sheet.

The transparent sheet is then placed over Fig. 17(a), disregarding the origin

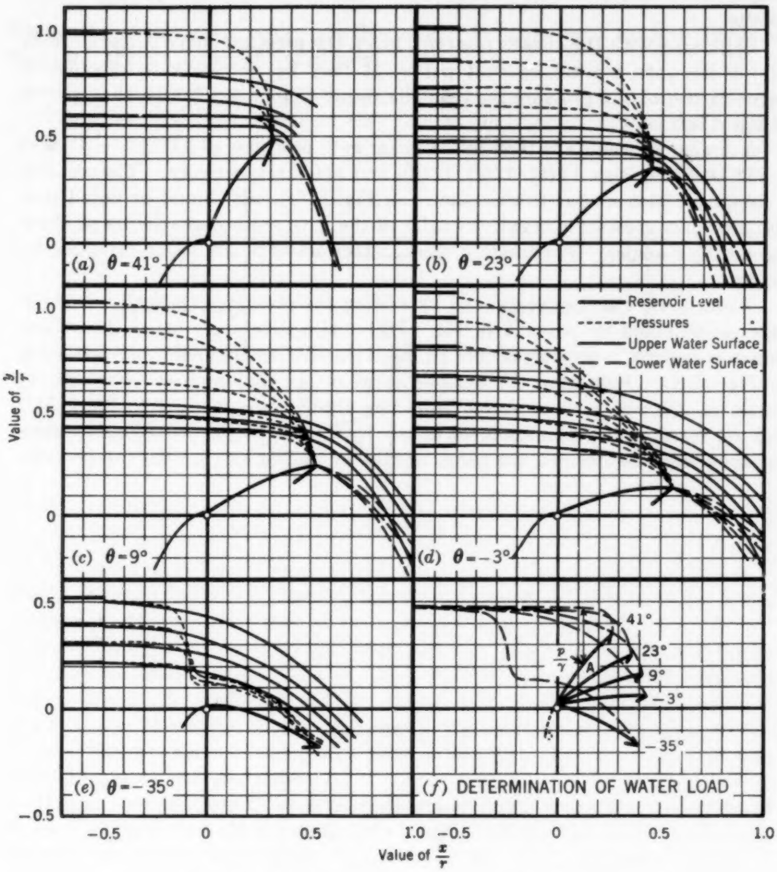

FIG. 17.—PRESSURE AND WATER-SURFACE PROFILES

of coordinates, and matching only the downstream tips of the two gates. The downstream part of all drum gates, regardless of size or radius, will coincide for any given value of θ. The height of the gate, or length of arc, can be expected to vary; this will have a negligible effect on pressures or water-surface profiles in the majority of cases. Should the gate under investigation differ from the height shown in Fig. 17(*a*), a small increase or decrease in the approach-depth results.

Beginning with the chosen reservoir level, the pressure curve is traced from Fig. 17(*a*) onto the transparent paper. It may be necessary to interpolate between two of the pressure curves. The result will be similar to that shown in Fig. 17(*f*).

A similar procedure is then followed for gate positions of 23°, 9°, −3°, and −35°, utilizing Figs. 17(*b*), 17(*c*), 17(*d*), and 17(*e*), respectively. The result is a composite plot similar to that shown in Fig. 17(*f*). It should be noted that the pressures shown for negative angles of the gate are not as reliable as those for positive angles. Fortunately, the greater water loads occur for positive angles.

Water loads can be determined by scaling the pressures vertically over the gate as indicated by point A in Fig. 17(*f*). If a gate angle other than those shown is desired, interpolation can be made directly on the sheet corresponding to Fig. 17(*f*). Following the establishment of the maximum-pressure curve, values of x/r and y/r are scaled from the sheet corresponding to Fig. 17(*f*) and are transferred to dimensional values by multiplying by r. Should water-surface profiles be desired, the same method of tracing and scaling can be used.

DISCUSSION OF
DESIGN OF SIDE WALLS IN CHUTES AND SPILLWAYS
PROCEEDINGS-SEPARATE NO. 175

LELAND S. RHODES.[3]—Some excellent points have been brought out by Mr.

[3] Prof. of Civ. Eng., Pennsylvania State College, State College, Pa.

Gumensky. However, under the heading, "Pressure in a Steeply Inclined Stream," there appears the statement that

> "The water, which is supported on a slope, has a negligible shearing value. Therefore, the floor supports only the normal component of the weight of the water."

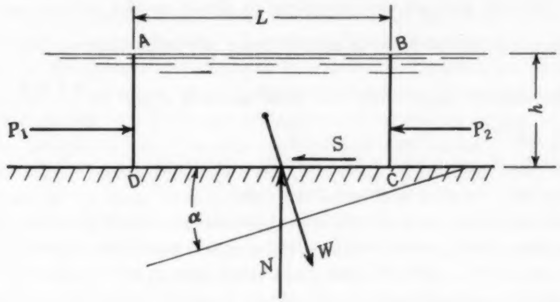

FIG. 7

This is not completely true, because the shearing force in a flowing liquid is not a negligible quantity, but may be even greater than the normal force.

Considering as a free-body the water in Fig. 7, labeled ABCD, the forces acting on the free-body are the two forces P_1 and P_2, which are the pressures of the surrounding water on ABCD, the weight of the water $W = \gamma\, b\, h\, L$, in which b is the width of the channel, N the normal pressure force of the channel bottom against the water, and S the shearing force between the channel surfaces and the water.

When the water begins to flow down the channel, the weight component $W \sin \alpha$ is larger than the shearing force S and therefore the water accelerates. Higher velocities produce higher values of S until, in a rather short length of channel, the velocity has reached a value at which $S = W \sin \alpha$, and there is equilibrium of forces and no further acceleration, since $P_1 = P_2$.

Since

$$N = W \cos \alpha = \gamma\, b\, h\, L \cos \alpha \qquad (8)$$

the pressure intensity on the floor is

$$p = \frac{N}{A} = \frac{\gamma\, b\, h\, L \cos \alpha}{b\, L} = \gamma\, h \cos \alpha \qquad (9)$$

as shown in Eq. 2*a*. The shearing force is

$$S = W \sin \alpha = \gamma\, b\, h\, L \sin \alpha \qquad (10)$$

and the unit shearing stress is equal to Eq. 10 divided by the area $(b + 2h)L$. If b is very large in comparison to $2h$ (a wide, shallow channel) then $s = \gamma\, h \sin \alpha$. This means that in addition to the normal water pressure on the bottom and sides, there is a tangential shearing force, which at $\alpha = 45°$ is equal to the normal force, and at steeper angles will exceed the normal force. As the angle α approaches 90° the normal pressure approaches zero, and the tangential shear approaches the weight of water. At 90° the flow can break loose from the channel, and the shear will drop to zero.

It is probable that a correct structural design should take into account both the normal pressure and the friction drag or shear on the bottom and sides.

Mr. Gumensky has defined ρ as the mass per unit volume; that is $\rho = \frac{\gamma}{g}$. Therefore, should not Eq. 4 show the total mass M equal to $\frac{\gamma\, h\, d\, A}{g}$, and Eq. 3 read $F_c = M\, \omega^2\, r$?

GURMUKH S. SARKARIA,[4] J.M. ASCE.—In static the water pressure intensity is uniform at the same depth below the surface and is the same in all directions. This can also be considered as being approximately correct for the slow laminar flow of water. Would this statement also apply to the flow of water down steep chutes or dam spillways? Also, would it be correct to consider the hydrodynamic pressure at a point on the concave vertical curves of spillway buckets as being equal in all directions?

[4] Asst. Design Engr., Punjab Irrig. Branch, Simla, India.

The total head or energy per unit weight at any point in a stream of water flowing down an inclined slope will be the sum of the pressure, the velocity heads, and the elevation. In case of gradually varied laminar flow, where the accelerative forces are negligible, the pressure head would be constant over any transverse section of the chute. This means that the pressure head would be equal at the side walls and center of the channel. This pressure head is determined from Eq. 2*a*. If the angle α is small, the pressure will be approximately equal to the product of γ and the vertical depth of water. This pressure, synonymous with hydrostatic pressure at a point, is equal in all directions and should be used for the design of the side walls. The velocity head is active on planes normal to the direction of flow and may be neglected when design pressure on the side walls is being considered.

The problem of flow along a concave vertical curve and the determination of the resultant pressures on the side walls differs from the problem of flow along a steep incline. This difference is caused by the fact that centrifugal force must be taken into consideration in the former problem. As shown in Fig. 3, the centrifugal force acts in a direction normal to the direction of the flow of water and increases the pressure on the bucket floor. Mr. Gumensky assumes that the pressure on the side walls will be increased by the amount of the centrifugal force. In other words, the author assumes that the velocity of the water along the walls is the same as at center of the chute and that the increase in hydrodynamic pressure, as a result of centrifugal force, will be the same in all directions.

The velocity of water flowing in a channel is not uniform across the cross section. It is a maximum near the center of the section; and at the sides of the section it is less than the average velocity of the section. This is also true for water flowing down vertically-concave steep chutes at velocities greater than the critical velocity. In such cases there will be secondary movements along the side walls, resulting in local eddies and a reduction in the velocity of water flowing along the walls. Thus, the centrifugal force of the water flowing along the walls will be smaller than that in the center of the channel.

The small centrifugal force along the walls will act in a radial direction on the bucket floor. There will be no hydrodynamic forces acting on the side walls unless there are acceleration components acting in the plane of the cross section or at right angles to the general direction of flow. This would be possible to some extent, if the cross section of the spillway chute or bucket were curved or concave instead of rectangular or trapezoidal. Any flow at right angles to the general direction of flow will exert additional hydrodynamic pressure on the side walls of such a chute in the form of centrifugal force.

By investigating Mr. Gumensky's approach to the inclusion of centrifugal forces in the design pressures on side walls of spillway buckets, the writer wants to illustrate the complex nature of the problem. Mr. Gumensky states that walls built along the sides of the curved bottom of a spillway chute, where the centrifugal force had not been included in the design, are near the point of failure. No walls are known to have failed because of the omission of the effect of centrifugal force. This shows that the inclusion of such a force in the design would result in a over-designed structure. Experimental observations of pressure on the walls of laboratory models would remove most of the doubts concerning the effects of centrifugal force. Such experimental results would be more reliable then deductions based on a theoretical approach to the prob-

J. H. Douma,[5] A.M. ASCE.—Basic design considerations of side walls for

[5] Hydr. Engr., Office of the Chf. of Engrs., Corps of Engrs., U. S. Dept. of the Army, Washington, D. C.

high-velocity chutes and spillways containing vertical curves are investigated by the author. Previous to this paper, incorrect design methods have been used, and in some cases the effect of centrifugal action has been completely neglected.

Although not specifically stated by Mr. Gumensky, centrifugal pressures occurring on the surface of the spillway bucket are equal to pressures occurring at the base of the spillway side walls. Bucket pressure must be considered in the stability analysis of the dam, and in the case of a buttress dam with a downstream spillway deck, the deck slab must be designed to withstand centrifugal pressures.

Because hydraulic loads resulting from centrifugal pressures are inversely proportional to the bucket radius, those loads can be held to reasonable limits by selecting the proper radius. For buttress dams it is desirable, in some cases, to select a longer radius than is needed to satisfy the hydraulic requirements, so that the spillway side walls and the downstream face deck will not be excessively thick.

Centrifugal pressures in a spillway bucket are computed by the expression,

$$p_c = \frac{\gamma\, q}{v} \frac{2\, h_1}{g} \qquad \text{(11a)}$$

which is obtained by substituting $d = \frac{q}{V}$ and $V = \sqrt{2\,g\,h_1}$ in the generalized equation for centrifugal pressure,

$$p_c = \frac{\gamma\, d\, v^2}{g\, r} \qquad (11b)$$

In Eqs. 11, p_c is the centrifugal pressure, in pounds per square foot; q denotes the discharge per foot of spillway, in cubic feet per second; h_1 is the head from the maximum reservoir level to a point on the bucket, in feet (Fig. 8(a)); and g represents the acceleration of gravity, in feet per second per second.

The advantage of using Eq. 11a instead of Eq. 5 is that values of h_1 can be determined more readily than the velocity. Although h_1 should be taken as the distance from the energy gradient to the bucket surface, the small head losses as a result of flow down the face of a dam may be neglected without appreciably affecting the thickness of the deck slabs or the side walls.

Eq. 11a yields variable centrifugal pressures because of variations in the value of h_1 throughout the bucket. The minimum and maximum centrifugal pressures in a bucket occur at the highest and lowest points of the bucket, respectively. The variation in centrifugal pressures is greater in a bucket of a low dam than in a bucket of a high dam because the relative variation in heads is greater for low dams.

Eq. 11a is applicable to roller-bucket stilling basins when h_1 is replaced by h_2, the head in feet, from the reservoir elevation to the tailwater elevation as in Fig. 8(a). The computed pressures will be approximate because the water-surface elevation of the bucket roller is seldom the same as the tailwater elevation. The approximate bucket pressure and the maximum side-wall pressures are obtained by adding the computed centrifugal pressures to the hydrostatic pressures corresponding to the tailwater depth.

Eqs. 11 are based on the assumption that streamlines instantly assume the paths dictated by the bucket radius at the beginning of curvature. The flow net theory indicates, however, that streamlines are influenced somewhat upstream of this point and continue to be influenced for some distance downstream of the point. A similar transition zone occurs at the downstream point of tangency. Therefore, actual pressure distributions in a bucket are realistically represented in Fig. 8(b), which is based on data obtained from model

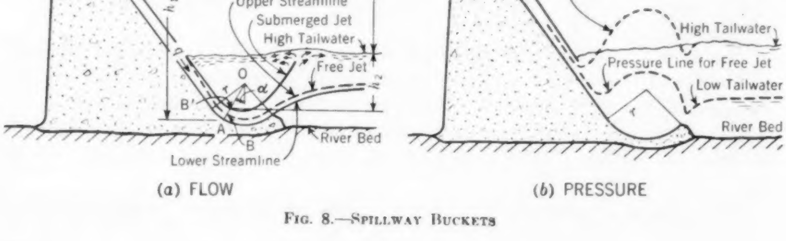

FIG. 8.—SPILLWAY BUCKETS

studies of Grand Coulee Dam, in Washington, Pine Flat Dam, in California, and Popolopen Dam at West Point, N. Y.

For a dam flow net, streamlines near the bucket boundary are circular; hence, the velocity is tangential at all points. These streamlines are determined by the function

$$\psi = c \log r = \text{constant} \qquad (12)$$

and equipotential lines are determined by

$$\phi = C\,\theta = \text{constant} \qquad (13)$$

The slope of the potential surface in the direction of flow equals the tangential velocity, and thus the absolute velocity is expressed by

$$V = \frac{d\phi}{dr} = -\frac{c}{r} \qquad (14)$$

When the flow occurs between two circular boundaries whose radii are r_1 and r_2 ($r_2 > r_1$), and the discharge between the two boundaries is q, then

$$q = \int_{r_1}^{r_2} V\,dr = \int_{r_1}^{r_2} \frac{c}{r}\,dr = -c\,(\log r_2 - \log r_1) \qquad (15a)$$

from which

$$c = -\frac{q}{\log (r_2/r_1)} \qquad (15b)$$

Substituting Eq. 15*b* into Eq. 13,

$$V = \frac{q}{r \log (r_2/r_1)} \qquad (16)$$

Referring to Fig. 8(*a*), if r_2 is the bucket radius and r_1 = OB, the velocity at point A for free-jet flow is

$$V_A = \frac{q}{r_2 \log (r_2/r_1)} \qquad (17)$$

At point B on the upper streamline the velocity is

$$V_B = \frac{q}{r_1 \log (r_2/r_1)} \qquad (18)$$

From Bernoulli's theorem, the pressure head at point A is

$$\frac{p}{\gamma} = h_1 - \frac{1}{2\,g}\left[\frac{q}{r_2 \log (r_2/r_1)}\right]^2 \qquad (19)$$

Since the pressure head at point B is zero, Bernoulli's equation becomes

$$h_1 - (r_2 - r_1)\cos\alpha - \frac{1}{2\,g}\left[\frac{q}{r_1 \log (r_2/r_1)}\right]^2 = 0 \qquad (20)$$

If values of h_1, α, q, and r_2 are known for any point, the corresponding values of r_1 may be determined by solution of Eq. 20. Then the pressure at the corresponding point on the bucket is determined from Eq. 19.

When the tailwater submerges the high-velocity jet, as in Fig. 8(*a*), the pressure on the upper streamline at point B is not zero but approximates the back-pressure caused by the tailwater. Then Eq. 20 becomes

$$h_1 - (r_2 - r_1) \cos \alpha - \frac{1}{2g} \left[\frac{q}{r_1 \log (r_2/r_1)} \right]^2 = h_2 \qquad (21)$$

and

$$H = h_1 - h_2 - (r_2 - r_1) \cos \alpha \qquad (22)$$

Solving Eqs. 21 and 22 simultaneously,

$$r_1 \log \left(\frac{r_2}{r_1} \right) = \frac{q}{\sqrt{2 g H}} \qquad (23)$$

From Eq. 23, values of r_1 may be computed if values of q, H, and r_2 are known. Corresponding bucket pressures are computed by use of Eq. 19.

A graphical solution for the pressure distribution in a spillway bucket, based on construction of the flow net, has been presented by Hunter Rouse,[6] M. ASCE. This solution is based on assumed flow conditions approaching the bucket curve. It is to be noted that the effect of the bucket curve on pressure extends beyond the two points of tangency.

[6] "Engineering Hydraulics," by Hunter Rouse, *Proceedings*, Fourth Hydraulics Conference, Iowa Inst. of Hydr. Research, John Wiley & Sons, Inc., New York, N. Y., 1950.

The methods presented by the author and the writer for determining pressures in spillway buckets and on side walls furnish conservative design loads when maximum computed pressures are used. The present (1953) state of knowledge is inadequate for safe reduction of design loads in the vicinity of the two points of tangency unless pressures in those areas are verified by model tests.

Generalized model tests are required to evaluate the following factors for flow in spillway buckets: (1) The approach flow conditions, (2) the upper water-surface profiles for free and submerged jets, and (3) the relationship between the tailwater depth and the hydrostatic bucket pressures for the submerged-jet condition. When these factors are determined in terms of head and discharge, Eqs. 18, 19, and 22 and the graphical solution can be modified to permit a precise determination of the pressures on side walls and upstream and downstream from, and within, spillway buckets.

EDGAR E. FOSTER,[7] M. ASCE.—Certain factors in the design of walls for

[7] Chf., Eng. & Cons. Div., Technical Cooperation for Iran, Teheran, Iran.

spillways, chutes, and similar structures have been clarified by the author. As he states, the functional design should be the first consideration; that is, the various dimensions and general layout should be fixed by the required hydraulic capacity. After that step the structural dimension can be fixed by the loads imposed.

The loads imposed by the flowing water, however, are not the only loads to be considered, except in the case of walls such as those shown for the spillway in Fig. 6. In many cases spillway and chute walls are backed by an earth fill which imposes loads that control the design because the walls are constructed to safe heights above the maximum water surface. Earth-imposed loads would have the greatest effect during periods of low water.

In general, it appears that small slopes would have little effect on altering either the water loads or earth-imposed loads. For steep slopes, such as those shown in Fig. 6, the effect would be great. It also seems logical to assume that steep slopes parallel to the wall would tend to diminish the earth-imposed loads in a manner similar to the decrease shown for water loads. It would be most helpful if this assumption were investigated.

THOMAS J. RHONE,[8] A. M. ASCE.—The author's analysis of the problem

[8] Hydr. Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

and his working equations are most commendable. It is apparent that an allowance should be made for the effects of centrifugal force, and the use of Mr. Gumensky's equations provides for such allowances.

When hydraulic model studies are made for an over-all evaluation of design, the side-wall pressures can be easily evaluated. Two examples of pressures in spillway side walls have been prepared from data obtained during model studies conducted in the hydraulic laboratory of the Bureau of Reclamation in Denver, Colo. The examples were taken from two different types of structures and indicate the conditions under which side-wall pressures may be investigated.

Example No. 1.—The data used in this example were obtained from hydraulic model studies made in 1936 on the stilling basin (a submerged roller bucket) for Grand Coulee Dam, on the Columbia River. The wall on which the pressure studies were made separates the stilling basin from the powerhouse tailrace and is completely submerged by the tailwater at the larger of the two discharges used in this example.

The pressure curves shown in Fig. 9 were derived by two methods: Curves A and C were computed from Eq. 6*a*, whereas curves B and D are pressures obtained from piezometers placed in the dividing wall. Curves A and B illustrate the conditions at a flow of 1,000,000 cu ft per sec and curves C and D show the conditions for a flow of 500,000 cu ft per sec. At a flow of 1,000,000 cu ft per sec, the water surface at the piezometers is at El. 997.5; and at 500,000 cu ft per sec, the water surface is at El. 975.2.

The measured pressures were taken from six piezometers placed normal to the face of the wall. The wall has a slight batter, but the pressure readings have been plotted in Fig. 9 on a horizontal line at the elevation of the piezometer. The pressures shown at the base of the wall were determined from a piezometer placed in the invert of the bucket rather than in the wall. The pressures computed from Eq. 6*a* also made use of model data; the velocity was

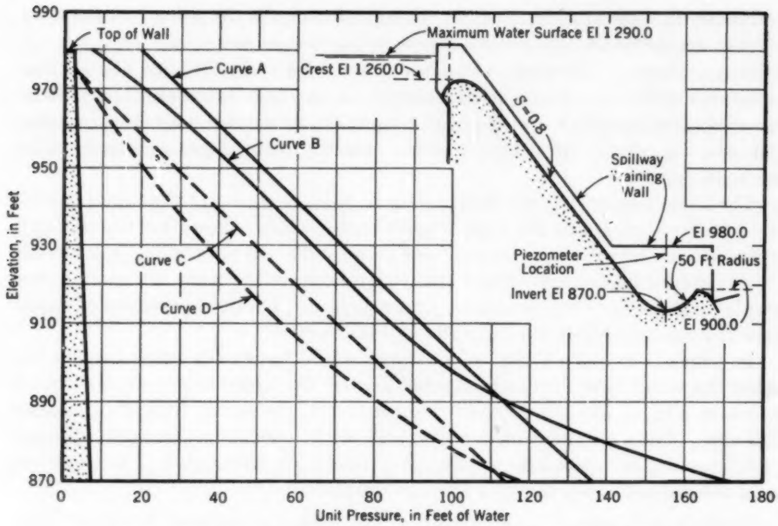

FIG. 9.—SIDE-WALL PRESSURES AT GRAND COULEE DAM STILLING BASIN

determined from pitot tube measurements; the valve used for r for pressure values near the wall base was equal to the radius of the bucket, but for other pressure values was equal to the approximate radius of the flow lines at the point under investigation.

The reverse flow near the water surface in the bucket was considered as having no velocity; therefore, there is no pressure caused by centrifugal force in the upper part of curves A and C.

The velocities in the submerged roller bucket (in a vertical plane directly over the invert of the bucket) were comparatively small—only 50 ft per sec at El. 870—and decreasing to 22 ft per sec at El. 910. With these velocities, the maximum pressure caused by centrifugal force was only 1.5 ft of water. Therefore, for all practical purposes, curves A and C represent the hydrostatic pressure.

In comparing the computed pressures with the measured pressures, the computed pressures for both discharges are greater than the measured pressures except near the base of the wall. For a discharge of 500,000 cu ft per sec, the computed and measured pressures are almost the same at the base of the wall. For a discharge of 1,000,000 cu ft per sec, the measured pressure is approximately 30% greater than the computed pressure. This difference may have been caused by the reflection of some impact head from the flow by the floor piezometer. However, the lowest of the wall piezometers (El. 885) also shows a pressure greater than the computed pressure (Fig. 9). Example No. 1 seems to indicate that the pressure resulting from the centrifugal force need not be considered when the flow becomes submerged with a resulting severe reduction in velocity.

Of further interest in this example is the fact that the measured pressures (over most of the range) are less than the hydrostatic pressures based on the

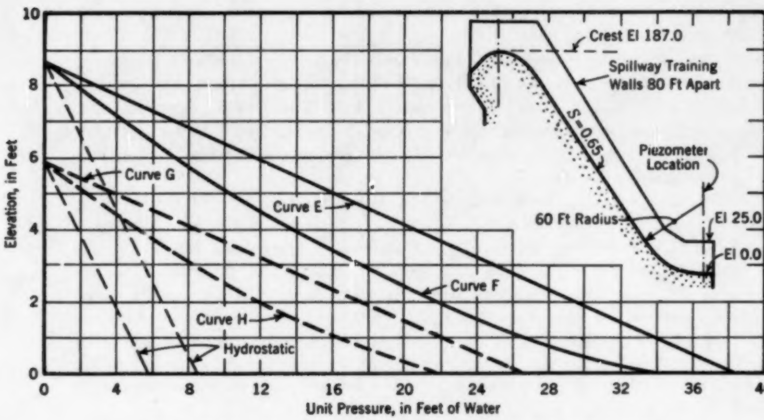

FIG. 10.—SIDE-WALL PRESSURES ON A 60-FT RADIUS FLIP BUCKET

given water-surface elevations. This difference may be due to air insufflation.

Example No. 2.—The data used in this example were obtained in 1953 from model studies performed on a high-head spillway having a 60-ft radius flip bucket. The bucket is not submerged but acts to change the direction of flow from the slope of the spillway face to the horizontal, as shown in Fig. 10. The flow discharges into the atmosphere between vertical training walls which contain piezometers.

Two discharges were also used in this example, and the wall pressures measured at the invert of the bucket are compared with the pressures computed from Eq. 6*a*. Curves E and G in Fig. 10 are the computed pressures, and curves F and H are the measured pressures. Curves E and F show the conditions at a flow of 56,100 cu ft per sec, and curves G and H illustrate the conditions for a flow of 35,600 cu ft per sec. At a flow of 56,100 cu ft per sec, the water surface at the piezometers is at El. 8.52; and at 35,600 cu ft per sec, the water surface is at El. 5.80. The velocity used in the computations is the average velocity obtained from

$$V = \frac{Q}{A} \qquad (24)$$

in which A is the product of the spillway width and the flow depth measured at the bucket invert. The value used for r in Eq. 6*a* is the radius of the flip bucket.

Fig. 10 shows that for both discharges the computed pressures are higher than the measured pressures for the full depth. A comparison of the pressures near the base of the wall shows that, at a flow of 56,100 cu ft per sec, the piezometer pressures and the computed pressures are, respectively, 4.0 and 4.5 times as great as the hydrostatic pressure. For a discharge of 35,600 cu ft per sec the piezometer pressures and the computed pressures are, respectively, 3.6 and 4.6 times as great as the hydrostatic pressure.

Example No. 2 shows that, although the computed pressures are higher than the measured pressures, they are not excessive. However, if the centrifugal force had not been included in the pressure value, the computed value would be only 25% of the actual force.

D. B. Gumensky,[9] M. ASCE.—Many thanks are due the engineers who

[9] Civ. and Structural Engr., Berkeley, Calif.

have shared with the profession some of their time, knowledge, and energy by contributing their discussions of the writer's paper. The discussions further elucidate the problem introduced in the paper. Theoretical deductions are supported by experimental measurements, new formulas for the computations of pressures in spillway buckets are offered, and some clarifying and critical observation are presented.

Mr. Rhodes, in Fig. 7 and in Eqs. 8 to 10, develops a value for the shearing stress of a sliding block ABCD. Eq. 10 is entirely applicable in a case of a solid block sliding down the surface. However, in the case of flowing water, most of the weight component parallel to the bottom of the channel is expended either in giving the water an acceleration or in imparting to the water a turbulence which consumes the energy as internal friction. The remarks of Mr. Rhodes regarding Eqs. 3 and 4 appear appropriate and are noted with gratitude. The slight confusion in these equations was caused by the use of the symbol F_c for both the unit centrifugal force in Eq. 3 and for the total centrifugal force for depth h in Eq. 5.

Mr. Foster acknowledges the necessary steps in the functional and structural design outlined by the writer and appropriately points out the need for the inclusion of loads imposed by earth fills. Such loads, especially in connection with earth dams, frequently control the structural design of the walls of water channels. The evaluation of lateral pressures caused by earth loads placed on sloping ground is somewhat complicated by the shearing strength of the earth which will vary between relatively simple cases of granular soils having internal friction but no cohesion, and difficult cases of highly cohesive clays which, in addition, may have expanding properties depending on their mineral composition.

Mr. Sarkaria discusses various phenomena connected with the flow of water on a slope and along concave surfaces in a bucket of a spillway. He indicates the complex nature of the problem by noting the uneven distribution of velocities in a flowing stream, and he questions the need for including in the design of bucket walls the increase in pressure caused by the centrifugal force. Mr. Sarkaria asks for experimental results rather than theoretical deductions. It appears that the experimental data given by Messrs. Douma and Rhone should satisfy Mr. Sarkaria's request.

Mr. Dc .ma calls attention to the necessity of considering centrifugal-force pressures in stability analyses and in the structural design of concrete gravity and buttress dams. This is a welcome reminder which could be extended into several particulars.

The use of Eq. 8*a* for the computation of centrifugal pressures in the bucket should be excellent in cases of low and moderate height dams. However, in cases of dams exceeding 150 ft in height the friction and turbulence losses probably would justify a more accurate, although a more tedious, analysis. The writer was impressed by the mathematical exposition of the relationship between pressures, velocities, heads, and radii in various parts of a discharge bucket as shown by Mr. Douma.

Figs. 8 are most valuable because they show the physical actuality of the behavior of a deflected water stream in a free jet and in a submerged jet. Much valuable information can be derived from the study of these figures.

The experimental data furnished by Mr. Rhone are a valuable contribution to the study of pressures in a spillway bucket. Mr. Rhone's discussion of these data is illuminating and furnishes a reliable tie between theoretical deductions and experimental measurements. In a case of a free jet the theory appears to be well supported by experiment.